AD-A106 336 SCHNABEL ENGINEERING ASSOCIATES RICHMOND VA F/G 13/13
NATIONAL DAM SAFETY PROGRAM. FAWN LAKE DAM (INVENTORY NUMBER VA--ETC(U)
JUL 81 R E MARTIN, C S ANDERSON, J G STARR DACW65-81-D-0020

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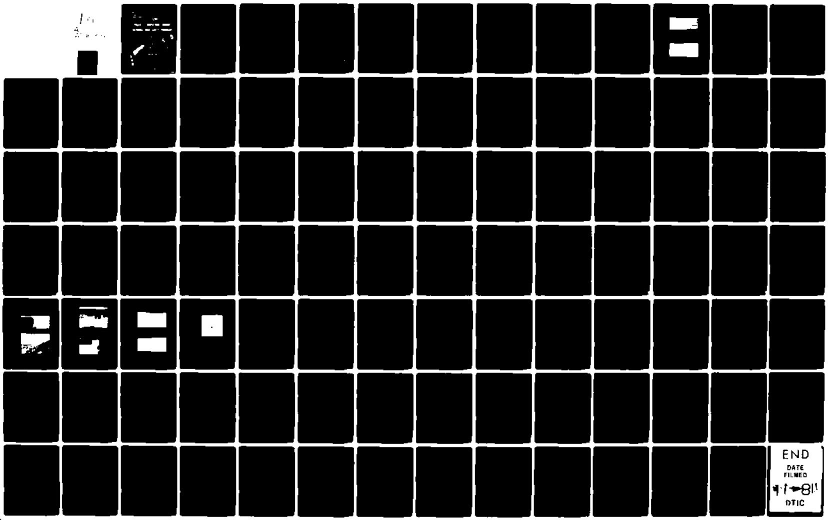

[illegible]

**Name Of Dam:** [illegible]

**Location** [illegible] COUNTY, VIRGINIA

**Inventory Number** VA [illegible]

AD-A[illegible]

LEVEL

# PHASE I INSPECTION REPORT
# NATIONAL DAM SAFETY PROGRAM







**PREPARED FOR**

NORFOLK DISTRICT CORPS OF ENGINEERS
803 FRONT STREET
NORFOLK, VIRGINIA 23510

**BY**

SCHNABEL ENGINEERING ASSOCIATES, P.C./
J. K. TIMMONS AND ASSOCIATES, INC.

JULY 1981

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20. Abstract

Pursuant to Public Law 92-367, Phase I Inspection Reports are prepared under guidance contained in the recommended guidelines for safety inspection of dams, published by the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I Inspection is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general conditions of the dam is based upon available data and visual inspection. Detailed investigation and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

Based upon the field conditions at the time of the field inspection and all available engineering data, the Phase I report addresses the hydraulic, hydrologic, geologic, geotechnic, and structural aspects of the dam. The engineering techniques employed give a reasonably accurate assessment of the conditions of the dam. It should be realized that certain engineering aspects cannot be fully analyzed during a Phase I inspection. Assessment and remedial measures in the report include the requirements of additional indepth study when necessary.

Phase I reports include project information of the dam appurtenances, all existing engineering data, operational procedures, hydraulic/hydrologic data of the watershed, dam stability, visual inspection report and an assessment including required remedial measures.

YORK RIVER BASIN

NAME OF DAM: FAWN LAKE DAM
LOCATION: SPOTSYLVANIA COUNTY
INVENTORY NUMBER: VA. NO. 17709

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM.

Fawn Lake Dam (Inventory Number VA 17709),
York River Basin. Spotsylvania County,
Virginia. Phase I Inspection Report.

9 Final rept.,

15 DACW65-81-D-0020

PREPARED FOR
NORFOLK DISTRICT CORPS OF ENGINEERS
803 FRONT STREET
NORFOLK, VIRGINIA 23510

12 14

BY

SCHNABEL ENGINEERING ASSOCIATES, P.C./
J. K. TIMMONS AND ASSOCIATES, INC.



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# PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D. C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through frequent inspections can unsafe conditions be detected and only through continued care and maintenance can these conditions be prevented or corrected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

BRIEF ASSESSMENT OF DAM

| | |
|---|---|
| Name of Dam: | Fawn Lake Dam |
| State: | Virginia |
| Location: | Spotsylvania County |
| USGS QUAD Sheet: | Chancellorsville, Virginia |
| Coordinates: | Lat 38° 15.5' Long 77° 42.9' |
| Date of Inspection: | April 20, 1981 |

Fawn Lake Dam is a homogeneous earthfill structure about 2230 ft long and 63 ft high. The principal spillway consists of a rectangular reinforced concrete riser (9 ft x 3 ft) and 240 ft of 36 inch diameter concrete discharge pipe. A 200 ft wide earthen emergency spillway is located at the left abutment, 3.5 ft above the principal spillway. The dam is an intermediate size structure and is assigned a significant hazard classification. The dam is located on Greenfield Creek, in Spotsylvania County, Virginia. The lake is used for recreation and is owned and maintained by the International Paper Realty Corporation.

Based on the criteria established by the Department of the Army, Office of the Chief of Engineers (OCE), the appropriate Spillway Design Flood (SDF) for the dam is the ½ PMF. The spillway will pass 45 percent of the Probable Maximum Flood (PMF) or 90 percent of the SDF without overtopping. During the SDF the dam will be overtopped by a maximum of 0.1 ft for a period of 1 hour at a maximum velocity of 1.7 fps. The spillway is judged inadequate, but not seriously inadequate.

The visual inspection did not reveal any problems which would require immediate attention. A better vegetative cover should be established in the emergency spillway. Several eroded areas existing

on the downstream slope should be examined during normal maintenance. Increased erosion will require local regrading and reseeding. Seepage areas described along the left and right downstream toe of the embankment should be monitored quarterly. Seepage along the right downstream toe occurs near the former stream channel and, therefore, should be carefully monitored to insure that increased flow or piping does not develop within the embankment. An emergency operation and warning plan should be developed. Furthermore, a staff gage should be installed to monitor water levels.

A summary of the design stability analysis of the upstream and downstream slopes under steady seepage conditions was reviewed and found to be acceptable. The factor of safety for rapid drawdown with respect to the upstream slope was less than one. The upstream slope does not meet the guidelines recommended by the Department of the Army, Office of the Chief of Engineers for earthfill dams subject to rapid drawdown. Consequently, it is recommended that any future lowering or draining of the lake be made at a rate of not more than 6 inches per day in order that a "rapid drawdown" condition does not develop. If this is not acceptable, it will be necessary to construct a properly designed toe berm.

SCHNABEL ENGINEERING ASSOCIATES, P.C./
J. K. TIMMONS & ASSOCIATES, INC.

Ray E. Martin, Ph.D., P.E.
Commonwealth of Virginia

Submitted by:

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Jack G. Starr, P.E.
Chief, Engineering Division

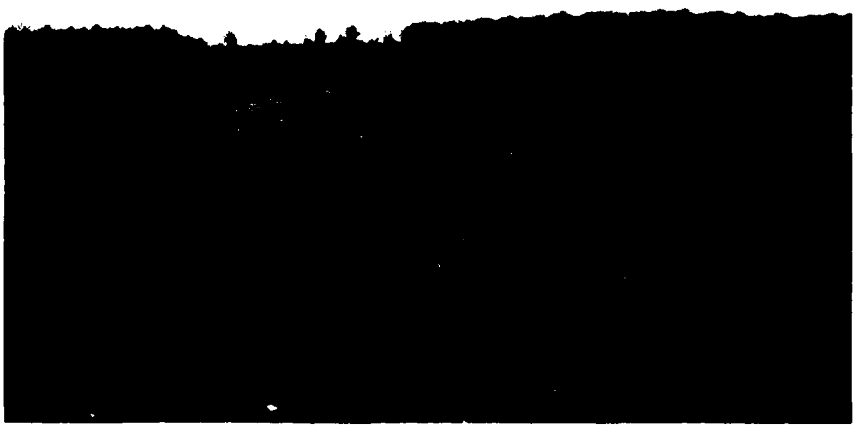
Approved:

Original signed by:
Douglas L. Haller

Douglas L. Haller
Colonel, Corps of Engineers
District Engineer

Date: AUG 6 1981

Lake



Dam

Overview Photographs

## SECTION I - PROJECT INFORMATION

1.1 General:

1.1.1 Authority: Public Law 92-367, 8 August 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of safety inspection of dams throughout the United States. The Norfolk District has been assigned the responsibility of supervising the inspection of dams in the Commonwealth of Virginia.

1.1.2 Purpose of Inspection: The purpose is to conduct a Phase I inspection according to the Recommended Guidelines for Safety Inspection of Dams (see Reference 1, Appendix V ). The main responsibility is to expeditiously identify those dams which may be a potential hazard to human life or property.

1.2 Project Description:

1.2.1 Dam and Appurtenances: Fawn Lake Dam is a homogeneous earthfill structure approximately 2230 ft long and 63 ft high.* The crest of the dam is 17 ft wide, and side slopes are approximately 2.5 horizontal to 1 vertical (2.5H:1V) on the upstream slope above normal pool and 3H:1V below normal pool, and 2.5H:1V on the downstream slopes of the dam. A 10 ft wide berm is shown on design drawings at elevation 336.5 msl along the upstream slope. The crest of the dam is at elevation 343 msl.

---

* Height is measured from the top of the dam to the downstream toe at the centerline of the stream.

The dam is keyed into the foundation and the ...

drainage system with a 6 inch drain outlet. ...

embankment slopes and riprap along the upstream ...

levels provide slope protection. (See Plate 9, Appendix I)

The principal spillway consists of a 9 ft x 3 ft ...

reinforced concrete riser inlet. The riser is ...

diameter reinforced concrete outlet pipe which runs ...

The riser crest elevation is 335.5 msl. A 30 inch diameter ...

gate is located 112 ft upstream of the riser on a 30 inch diameter

concrete inlet pipe at an invert elevation of 286.4 msl, and is used to

drain the lake. The outlet pipe is 240 ft long and the invert elevation

at the outlet structure is 281 msl. (See Plates 5, 7 and 8, Appendix I)

The emergency spillway (EMS) consists of a vegetated channel located at the left abutment, with a crest elevation of 339 msl. The emergency spillway is 200 ft wide, has 2H:1V side slopes and is in a cut section. The approach channel to the EMS is approximately 180 ft long at a 20% slope rising up to the control section. The discharge channel gently slopes away from the control section and intersects the stream approximately 500 ft downstream of the dam toe.

1.2.2 Location: Fawn Lake Dam is located on Greenfield Creek approximately 5.5 miles southwest of Chancellorsville, Virginia. (See Plate 1, Appendix I)

1.2.3 Size Classification: The dam is classified as an "intermediate" size structure based on its height and maximum lake storage potential.

1.2.4 Hazard [illegible]

area, however, based on the [illegible] located four miles downstream, the dam [illegible] hazard classification. The hazard [illegible] a dam is a function of location only and has [illegible] stability or probability of failure.

1.2.5 Ownership: The dam is owned and maintained by the International Paper Realty Corporation.

1.2.6 Purpose: Recreational.

1.2.7 Design and Construction History: The dam was designed and constructed under the supervision of Quible and Associates. The structure was constructed by Bishop and Settle and completed in 1975. Riprap was installed on the upstream slope in 1980 in accordance with design developed by J. K. Timmons & Associates, Inc.

1.2.8 Normal Operational Procedures: The principal spillway is ungated, therefore, water rising above the crest of the riser inlet is automatically discharged downstream. Normal pool is maintained at elevation 335.5 msl at the crest of the riser. Flood discharges which cannot be absorbed by storage and the riser, flow through the emergency spillway at pool elevations above 339 msl. The 30 inch diameter gate at elevation 286.4 msl is manually operated, and is used to lower the lake below normal pool.

1.3 Pertinent Data:

1.3.1 Drainage Area: The drainage area is 4.14 square miles.

[illegible] CFS.

[illegible] Spillway [illegible]:

[illegible] (Elev. 34[illegible]) [illegible] CFS

[illegible] Spillway [illegible]:

[illegible] at crest of dam (Elev. 343) 4.44 CFS

[illegible] Dam and Reservoir Data: See Table 1.1, below:

TABLE 1.1 DAM AND RESERVOIR DATA

| Item | Elevation feet msl | Reservoir: Area Acres | Reservoir: Storage Volume Acre Feet | Reservoir: Storage Watershed Inches | Reservoir: Length Miles |
|---|---|---|---|---|---|
| Crest of Dam | 343 | 352 | 7472 | 34 | 1.8 |
| Emergency Spillway Crest | 339 | 315 | 6084 | 27.5 | 1.6 |
| Principal Spillway Crest | 335.5 | 285 | 5113 | 23.16 | 1.4 |
| Streambed at Down-stream Toe of Dam | 279.8 | - | - | - | - |

[illegible] DATA

[illegible]: The dam was designed by and constructed under the [illegible] of Quible and Associates (Chase City, Virginia), [illegible] for American Central Corporation. Actual construction was performed by Bishop and Settle Construction Company, Inc., of [illegible], Virginia. Design drawings and reports are available from the International Paper Realty Corporation.

The dam was designed to accommodate a 50-year rainfall. Rainfalls of greater magnitude than the 50-year occurrence will discharge through a 200 ft wide emergency spillway in addition to the principal spillway. Design was based on future runoff conditions in accordance with development plans for the drainage basin. The design highwater for the lake permits a 3 ft freeboard.

The principal spillway was designed as a drop inlet structure consisting of a reinforced concrete riser, a transition section at the base of the riser, a 36 inch conduit and a plunge pool at the outlet end of the conduit. Seven reinforced concrete anti-seep collars spaced at 24 ft intervals were installed around the principal spillway pipe upstream of the drain trench in order to control any potential piping problems along the pipe. Details of the principal spillway and riser are presented on Plates 5 and 8 of Appendix I.

A subsurface investigation was conducted at the site by Gnaedinger, Baker, Hampton and Associates during the initial design stages. The investigation consisted of drilling seven test borings along the center line of the dam, two probe holes along the pipe line and eight auger

holes within the borrow areas. Test boring logs and associated subsurface profiles for borings made along the center line of the dam area are shown on Plate 4, Appendix I. The geotechnical report, with recommendations, was prepared based upon the test boring and laboratory test data. The text of this report is included as Appendix IV.

The dam as designed is a homogeneous, compacted-fill earth embankment constructed from ML, CL, SM and SC materials derived from designated borrow areas and the emergency spillway. A thin layer of topsoil was required to cover the embankment. Details of the dam and emergency spillway are provided on Plate 3, Appendix I. A typical section of the embankment and construction specifications for embankment fill materials are presented on Plate 5, Appendix I. A toe drain (i.e., drainage blanket) consisting of permeable filter material over a width of 30% of the dam height was recommended in the design report for the downstream toe. Onlyone toe drain consisting of 6" drainpipe as indicated on Plate 6, Appendix I was constructed beneath the downstream slope.

The design report recommended that the core and embankment fill be placed in 9-inch layers and compacted to at least 90% of maximum dry density in accordance with ASTM D-1557, Modified Proctor. Specifications for the fill work provided on Plate 5, Appendix I appear to generally comply with this recommendation.

Design drawings show the dam being founded on overburden soils and includes a 15 ft wide cutoff trench, which extends the length of the dam axis. This trench has 1H:1V design side slopes and it was

recommended by the geotechnical consultant that, "Material used to construct the core should be obtained from Borrow Area 3. This material should contain as much clay as possible." Details are given on Plate 4, Appendix I. The design report stated that if a positive cutoff were required, it was necessary to extend the core at least 10 ft into sound rock. On the other hand, if a positive cutoff was not required, then it was recommended that the base of the core trench extend a minimum depth of 20 ft below the base of the dam. The drawings give a planned depth for the core trench of about 10 to 20 ft. We understand that portions of the cutoff subgrade were observed by Quible and Associates, but that no written record of the subgrade materials is available.

The emergency spillway was designed as a trapezoidal channel and is located adjacent to the left abutment. It was formed by making a cut across a broad hill into residual soils. These soils appeared to consist of SM, ML and CL materials with assorted amounts of gravel. The crestline of the 200 ft wide spillway is at elevation 339 msl, an elevation which would have a frequency of use less than once in 50 years.

A design stability analysis was performed by Gnaedinger, Baker, Hampton and Associates. Details of this analysis are presented on pages 7, 8 and 9 of the Design Report (Appendix IV).

Effective strength parameters obtained on compacted samples were as follows:

$$\phi' = 26^\circ \qquad \bar{c} = 9 \text{ psi}$$

$$\phi' = 30^\circ \qquad \bar{c} = 6.5 \text{ psi}$$

Stability analyses were made for steady state and sudden drawdown conditions for water at maximum pool level. All factors of safety prove adequate except a toe failure condition of the upstream embankment during sudden drawdown. A toe berm along the upstream face was recommended in the report but was not provided. The design drawings indicate no berm was provided.

2.2 Construction: The construction records were not included with the information provided by the Owner, and only incomplete records were obtained from Quible and Associates. No records of the actual embankment materials used, compaction of fill attained, or observations of cutoff trench subgrade materials were provided.

During a 1979 dam inspection requested by the owner, considerable erosion at pool level was observed. It was recommended that the riprap be placed on the upstream slope to provide slope protection. In compliance with this recommendation, J. K. Timmons and Associates, Inc., were contracted for design of the riprap (See Plate 9, Appendix I). The riprap was installed in the summer of 1980.

2.3 Evaluation: Design drawings are representative of the structure and hydrologic and hydraulic calculations were available. There is sufficient information to evaluate foundation conditions and embankment stability. Recommendations made by the geotechnical consultant, which were not included in the design drawings, are as follows:

1) No riprap was placed along the upstream slope at pool level.
2) A toe berm was not shown for the upstream slope.
3) A drainage blanket was not provided beneath the embankment. Instead a toe drain was constructed.

## SECTION 3 - VISUAL INSPECTION

3.1 Findings: At the time of inspection, the dam was in excellent condition. Field observations are outlined in Appendix III.

3.1.1 General: An inspection was made on April 20, 1980. The weather was cloudy and windy and the temperature was approximately 55°F. Light rain had occurred earlier in the morning prior to the inspection. Ground conditions were wet at the time of the inspection. The pool and tailwater levels at the time of the inspection were 332 and 280 msl, respectively. The lake level was approximately 3.5 ft below normal. A Phase I type inspection was performed for the owner by Schnabel Engineering Associates, P.C./J. K. Timmons & Associates, Inc., on June 5, 1979.

3.1.2 Dam and Spillway: The crest and slopes of the embankment are grassed and well maintained. Vegetation along the emergency spillway was also well maintained but sparse. Several eroded areas exist along the upstream face below the riprap due to wave action and lack of vegetation. Two stabilized eroded areas approximately 20 ft wide and 1 ft deep were observed on the downstream slope to the left of the principal spillway outlet. Both areas were grass covered. Scattered erosion approximately 1 ft wide and 1 to 2 ft deep was also noted along the right downstream abutment. This erosion appears to be caused by runoff. Scattered shallow washes or eroded areas also exist along the downstream slope and within the lower portions of the emergency spillway.

The principal spillway riser, outlet pipe and plunge pool did not reveal any signs of deterioration. The drain valve on the intake structure was in good operating condition according to Mr. E. Webb, since it has been in use to control pool levels during recent repair operations. A six inch toe drain outlet is located to the right of the outlet pipe. Flow from the toe drain was clear and estimated between 2 - 4 gpm. This is similar to flows observed in the 1979 inspection.

A continuous wet area approximately 1000 ft in length occurs along the left downstream toe (see Field Sketch, Sheet 1, Appendix III). Scattered clusters of cattails and marsh grass are present in this area. No flow was observed except where natural sloping ground conditions exist. The water was generally clear and little or no iron staining was observed. This area was of initial concern during the 1979 inspection. During that inspection several hand auger holes were drilled along the left toe of the dam near the cemeteries, beside wet areas. These holes did not fill with water and it was, therefore, concluded that water was not passing through or under the embankment at those locations. At other nearby locations water was standing at elevations estimated several feet above and beyond the toe of the dam. Wet conditions are probably related to springs which were reportedly in existence prior to construction.

Seepage was also observed along the right downstream toe in the same area it was encountered in the 1979 inspection (see Field Sketch, Sheet 1, Appendix III). This area reportedly bounds the former stream channel. Essentially no erosion was encoutnered on the embankment. The presence of dead trees make this area suspect as to whether this seepage is related to previously existing springs or in fact related

to seepage under the embankment along the former stream channel. The water appeared clear and little or no iron staining was observed. No flow was observed.

3.1.3 Reservoir Area: The reservoir area was free of debris and the perimeter was wooded on all sides (Overview Photograph, Page 3). The reservoir is located in a natural valley with side slopes at approximately 4H:1V. No sediment build-up was observed.

3.1.4 Downstream Areas: The downstream channel showed no erosion. Patches of cattails were scattered throughout the channel. The channel is approximately 3 ft deep and 20 ft wide with 2H:1V side slopes. Along either side of the channel is a broad floodplain with gentle slopes. It is heavily wooded beyond the intersection with the emergency spillway and discharge channel. The floodplain is approximately 500 ft wide. Two dwellings located on Virginia Route 649 at the Po River approximately 4 miles below the dam are situated within the floodplain area.

3.1.5 Instrumentation: No instrumentation (monuments, observation wells, piezometers, etc.) was encountered for the structure. A staff gage was not observed.

3.2 Evaluation: Overall, the dam appeared to be in good condition at the time of the inspection.

3.2.1 Dam and Spillway: The vegetative cover on the embankment and emergency spillway appeared to be well maintained. Vegetation in the emergency spillway is sparse and reseeding is recommended to restrict surface erosion. Erosion on the upstream face below the riprap is the result of below normal lake levels. This erosion is not considered a

problem unless lake levels remain low and the riprap begins to be undermined. Erosion observed along the right downstream slope-abutment contact and also to the left of the principal spillway outlet should be monitored. Increased erosion will require local regrading and reseeding. The scattered shallow washes noted on the embankment and in the emergency spillway do not inhibit the satisfactory performance of the dam and, therefore, no special attention is recommended.

The water observed along the right downstream toe is of concern, since it could represent seepage along the former streambed. If this is the case, the potential for piping and embankment failure exists. The seepage should be monitored quarterly for increased flows or turbidity. If these conditions should occur, a Professional Engineer with expertise in Geotechnical Engineering should be contacted to evaluate the problem and make recommendations for required corrective measures. Water encountered along the left downstream toe beside the cemeteries appears to be related to spring activity. No corrective measures appear necessary at this time, but this area should also be examined periodically to insure that migration of saturation and also erosion of the downstream toe do not occur in this area. Seepage conditions observed during this inspection appeared to very similar to conditions noted in the 1979 inspection. The principal spillway is functioning well. A staff gage should be installed to monitor water levels.

3.2.2 Downstream Area: A breach in the Fawn Lake Dam during extreme flooding could create a hazard to the downstream dwellings.

## SECTION 4 - OPERATIONAL PROCEDURES

4.1 Procedures: The normal storage pool is elevation 335.6 msl or 0.1 ft above the crest of the concrete principal spillway drop-inlet riser. The lake provides recreation as its principal use. Water passes automatically through the principal spillway as the water level in the reservoir rises above the elevation 335.6 msl. Water will also pass automatically through the emergency spillway when the water level in the reservoir reaches elevation 339 msl. A 30 inch slide gate valve at the low point in the riser structure is provided to drawdown the reservoir from normal pool.

4.2 Maintenance of Dam and Appurtenances: Maintenance is the responsibility of the owner. Maintenance consists of inspection, debris removal, mowing of vegetative cover, and repair. Maintenance is routinely performed.

4.3 Warning System: At the present time, there is no warning system or evacuation plan for the dam.

4.4 Evaluation: The dam and appurtenances are in good operating condition, and maintenance of the dam appeared to be adequate.

An emergency operation and warning plan should be developed. It is recommended that a formal emergency procedure be prepared and furnished to all operating personnel. This should include:

a. How to operate the dam during an emergency.

b. Who to notify, including public officials, in case evacuation from the downstream area is necessary.

## SECTION 5 - HYDRAULICS/HYDROLOGIC DATA

5.1 Design: Fawn Lake Dam was designed as a single purpose dam, and hydrologic and hydraulic data are available, including stage-discharge, stage-storage, stage-area, inflow hydrograph and flood routing data.

5.2 Hydrologic Records: There are no records available.

5.3 Flood Experience: According to Mr. Webb, there have been no significant floods since construction of the dam.

5.4 Flood Potentials: In accordance with the established guidelines, the Spillway Design Flood (SDF) is based on the estimated "Probable Maximum Flood" for the region (flood discharges that may be expected from the most severe combination of critical meterologic and hydrologic conditions that are reasonably possible in the region), or fractions thereof. The Probable Maximum Flood (PMF) and ½ PMF and 100 year flood hydrographs were developed by the HEC-1 method (Reference 4, Appendix V). Precipitation amounts for the flood hydrograph of the PMF and 100 year flood were taken from U. S. Weather Bureau Information (Reference 5 and 6, Appendix V). Appropriate adjustments for basin size and shape were accounted for. These inflow hydrographs were routed through the reservoir to determine maximum pool elevations.

5.5 Reservoir Regulations: For routing purposes, the pool at the beginning of flood was assumed to be at elevation 335.6 msl. Reservoir stage-storage data and stage-discharge data were determined

from the [illegible] report and [illegible] for pool elevations up to 339 msl. Above pool elevation 339 msl stage-storage data was computed from construction plans, and stage-discharge data was computed for the emergency spillway and principal spillway. Floods were routed through the reservoir using the principal spillway discharge up to a pool storage elevation of 339 msl and a combined principal and emergency discharges for pool elevations above 339 msl. Discharges above pool elevations 343 msl were routed over the non-overflow section of the dam in addition to the principal and emergency spillways.

5.6 Overtopping Potential: The predicted rise of the reservoir pool and other pertinent data were determined by routing the flood hydrographs through the reservoir as previously described. The results for the flood conditions PMF, ½ PMF and 100 year flood are shown in the following Table 5.1:

TABLE 5.1 - RESERVOIR PERFORMANCE

| | Normal Flow | Hydrograph: 100 Yr. Flood | ½ PMF | PMF |
|---|---|---|---|---|
| Peak Flow, CFS | | | | |
| Inflow | 4 | 5015 | 12,473 | 24,94[illegible] |
| Outflow | 4 | 387 | 4,694 | [illegible] |
| Maximum Pool Elevation Ft, msl | 335.6 | 339.3 | 343.1 | 344.[illegible] |
| Non-Overflow Section (Elev 343 msl) | | | | |
| Depth of Flow, Ft | - | - | 0.1 | 1.9 |
| Duration, Hours | - | - | 1.0 | 4 |
| Velocity, fps | - | - | 1.7 | [illegible] |
| Tailwater Elevation Ft, msl | 280.2 | 282 | 283 | [illegible] |

5.7 <u>Reservoir Emptying Potential</u>: A 30 inch diameter gate at elevation 284 msl is capable of draining the reservoir through the outlet culverts. Assuming that the lake is at normal pool elevation (335 msl) and there is 4 cfs inflow, it would take approximately 5 days to lower the reservoir to elevation 284 msl. This is equivalent to an approximate drawdown rate of 10 ft/day based on the hydraulic height measured from normal pool to the invert of the drawdown pipe divided by the time to dewater the reservoir.

5.8 Evaluation: The U. S. Army, Corps of Engineers' guidelines indicate the appropriate Spillway Design Flood (SDF) for an intermediate size, significant hazard dam is the ½ PMF to PMF. Because of the risk involved, the ½ PMF has been selected as the SDF. The spillway will pass 45 percent of the PMF without overtopping the crest of the dam (90 percent of the SDF). During the SDF the dam will be overtopped by a maximum of 0.1 ft for a period of 1 hour at a maximum velocity of 1.7 fps.

Hydrologic data used in the evaluation pertains to present day conditions with no consideration given to future development.

## SECTION 6 - DAM STABILITY

6.1 Foundation and Abutments: The dam site is located within the Piedmont Physiographic Province of Virginia. The dam appears to be founded on residual soils derived from the in-place weathering of the underlying bedrock. The Geologic Map of Virginia indicates the bedrock consists of metamorphosed volcanic to sedimentary rocks and may include intrusions of younger igneous rocks. No rock outcrops were encountered during the inspection, and examination of residual soils did not reveal the presence of any relict bedrock structure. Scattered rock debris in the emergency spillway suggests that the site is underlain in part by mica schist bedrock and numerous quartz veins.

The geotechnical consultant described the soil profile along the centerline of the dam as varying considerably. The general profile reportedly consists of an upper clay layer overlying silt, which in turn overlies weathered rock. Between the right abutment and just beyond the principal spillway (Station 13+00), the upper layer of silty clay (CL) ranged from 3 to 4 ft in thickness. The clay layer increased in thickness to about 8 ft in the left abutment. The underlying silts and clayey silts (ML) contained various combinations of sand, gravel, and mica. The test borings indicated the presence of a very irregular bedrock surface. Bedrock (gray sandy shale) was encountered at 9.5 ft along the former stream channel, but was not encountered in the left abutment to a depth of 100 ft (B-7). Disintegrated rock, however, was present at depths of 60 to 100 ft in this area. Except for the previously described shale, the disintegrated

rock and bedrock consisted of different varieties of mica schist and apparently included scattered quartz veins.

The potential for seepage within the foundation was recognized by the geotechnical consultant, since the construction of a core trench was recommended in the design report. The general physical character of the overburden, i.e., clayey silt and silts consisting of various combinations of sand and rock fragments will usually allow seepage to occur. Low core recoveries suggest that the top of bedrock is generally weathered and fractured, and therefore, susceptible to seepage. In situ permeability tests made in four of the seven test borings indicated the permeability condition of the materials encountered was variable, probably from medium-high to low. Based upon the presence of these materials the design report stated that if a positive cutoff were required, the core should extend at least 10 ft into sound rock. If not, the core should extend a minimum depth of 20 ft below the base of the dam.

Gradual consolidation of underlying soils would be expected during application of fill materials. The underlying soils probably had essentially fully consolidated under the applied load not long after completion of construction. Based upon the performance history of this dam and the materials present, a stable foundation is assumed.

6.2 Embankment:

6.2.1 Materials: The upstream slope is 2.5H:1V with crest at elevation 343 msl. At approximately elevation 336.5 msl, the slope flattens to 10H:1V forming a berm for a vertical distance of 1 ft. The slope then continues at 3H:1V to natural ground. Normal pool level is elevation 335.5 msl or 1 ft below the top of this berm. The design

upstream slope does not include a toe berm as recommended by the geotechnical consultant, which would give the required factor of safety for the sudden drawdown condition. The downstream slope is 2.5H:1V. A 15 ft wide sloping core trench with 1H:1V side slopes is provided beneath the embankment. A typical section of the dam is included on Plate 5, Appendix I. The core and embankment material is designated Zone A and is specified to consist of "plastic or semi-plastic silts and clays." A thin blanket of topsoil or Zone B material was specified as cover for the Zone A soils. Although two zones are identified, for stability purposes this dam is considered a homogeneous structure. Fill material descriptions and compaction requirements are also given on Plate 5, Appendix I.

6.2.2 Subdrains and Seepage: A drainage blanket covering the downstream area in width equal to at least 30% of the dam height was recommended by the geotechnical consultant. However, the design drawings indicate that only a toe drain was provided to control the phreatic level through the embankment. Drainage details are provided on Plate 6 of Appendix I. The six-inch drain was discharging 2 - 4 gpm of clear flow during the inspection.

No flowing seepage was observed, however, saturated areas supporting marsh grass and cattails were observed along the downstream toe of the embankment. Saturated conditions were encountered along the left downstream toe from the northernmost cemetery extending southward toward the outfall structure (see Field Sketch 1, Appendix III). Moisture or seepage appeared to be confined to the area occupied by marsh grass and to areas immediately to the east. The water often appeared stagnant and only seemed to flow in definite sloping areas.

Little or no iron staining was observed in this area. Wet areas are probably related to springs which reportedly existed prior to construction.

Moisture or seepage was again encountered along the right downstream toe, but only between Stations 21+00 and 19+00, where the seepage path bends southward along the woodline. Marsh grass, cattails and dead trees are present in this area. The seepage is either related to previously existing springs or seepage under the embankment along the former stream channel.

6.2.3 Stability: The dam is 63 ft high and has a crest width of approximately 17 ft. Side slopes are 2.5H:1V on the downstream side, and 2.5H:1V to 3H:1V on the upstream side. A 10 ft wide berm exists on the upstream slope at approximately El 336.5, separating the slightly steeper upper portion of the embankment from the less steep lower portion.

Design stability analyses (see p. 7-9, Appendix IV) were performed by the geotechnical consultant for steady state and rapid drawdown conditions for water at maximum pool level. The Bishop method of slices was used for the analysis. Two triaxial tests were performed using on-site soils proposed for use as embankment fill. The effective strength paramters obtained are as follows:

$$\phi' = 26^\circ \qquad \bar{c} = 9 \text{ psi}$$

$$\phi' = 30^\circ \qquad \bar{c} = 6.5 \text{ psi}$$

For the upstream slope, the factors of safety for a deep circle, slope stability failure for water at maximum pool level and steady state conditions was approximately 2.5. A factor of safety of approximately 1.55 was calculated for rapid drawdown. Again for the upstream slope,

analysis for a toe failure for water at maximum pool level and steady state conditions indicates a factor of safety of approximately 2.3. A factor of safety of less than 1 was calculated for the rapid drawdown condition. For the downstream side of the dam, the analysis for a toe failure slide indicates a factor of safety of 4.

6.2.4 Seismic Stability: The dam is located in Seismic Zone 2. Therefore, according to the Recommended Guidelines for Safety Inspection of Dams, the dam is considered to have no hazard from earthquakes provided static stability conditions are satisfactory and conventional safety margins exist.

6.3 Evaluation: An accurate check on the stability of this structure can be made from the available design data (Appendix IV). The data reviewed were found to be acceptable. The stability analysis for the downstream slope under steady seepage conditions indicates a factor of safety of 4.0. This value exceeds the recommended factor of safety of 1.5 included in the Recommended Guidelines for Safety Inspection of Dams, Reference 1, Appendix V. For the upstream slope a factor of safety of 1.55 was calculated for a deep circle failure for rapid drawdown. This exceeds the recommended factor of safety of 1.2 included in Reference 1, Appendix V. A value of less than 1.0 was reported, considering a toe failure under the rapid drawdown condition. This value is less than that recommended in Reference 1. Factors of safety of 2.5 and 2.3 were recorded for upstream steady state conditions assuming deep seated and toe failures, respectively.

Although no undue settlement cracking, or seepage was noted at the time of the inspection, and the embankment appears adequate for maximum pool level with water at elevation 339.9 msl, toe failure of the upstream slope could occur if the water level is drawn down rapidly. Since the factor of safety for steady state conditions (water at normal pool) is very high at 2.5, we believe that considerable lowering of the lake at a rapid rate would be required to lower the factor of safety to that approaching failure conditions. Assuming these circumstances, although damage to the embankment would occur, the risk of detrimental downstream flooding due to the rapid drawdown condition is not likely since (1) the lake level at the time of failure would be very low and the volume of water greatly reduced, and (2) a breach in the dam through nearly its entire depth would be necessary for the water to discharge. Nevertheless, because the maximum rate of drawdown which is possible through the principal spillway is considered "rapid" for the type of embankment soils used, we recommend as a precaution that any draining or lowering of the lake level be done such that the rate of lowering does not exceed 6 inches per day to preclude any possible upstream toe failure. If it is anticipated that this rate must be exceeded in the future, or cannot be tolerated, a berm on the upstream toe must be provided in accordance with an acceptable design.

Overtopping is not considered detrimental to the dam with respect to erosion because of the shallow depth and short duration of flood. Also the velocity is considerably less than 6 fps, the effective eroding velocity for a vegetated earth embankment.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 Dam Assessment: Fawn Lake Dam at the time of inspection appeared to be in excellent condition. The appropriate SDF for this dam is the ½ PMF. The spillway will pass 45 percent of the PMF (90 percent of the SDF) without overtopping the crest of the dam. Flows overtopping the dam at 1.7 fps during the SDF are not considered detrimental to the embankment with aspect to erosion. The spillway is rated inadequate, but not seriously inadequate.

The actual embankment structure appears to be simlar to the "design" drawings. However, the design did not incorporate an upstream toe berm to provide an adequate slope safety factor for the rapid drawdown case. Although the design factor of safety for steady seepage meets the requirement of Reference 1, Appendix V, the factor of safety for rapid drawdown of the upstream embankment is inadequate. The stability of the dam is considered adequate for its present condition and at normal pool level, but inadequate for the rapid drawdown condition.

7.2 Recommended Remedial Measures:

7.2.1 Emergency Operation and Warning Plan: It is recommended that a formal emergency procedure be prepared, prominently displayed, and furnished to all operating personnel. This should include:

1) How to operate the dam during an emergency.
2) Who to notify, including public officials, in case evacuation from the downstream is necessary.

7.2.2 Any future lowering or draining of the lake should be at a rate of less than 6 inches per day in order that a "rapid draw-down" condition does not develop. If this is not acceptable, it will be necessary to construct a properly designed toe berm for the upstream slope.

7.3 Required Maintenance:

7.3.1 A staff gage should be installed to monitor water levels.

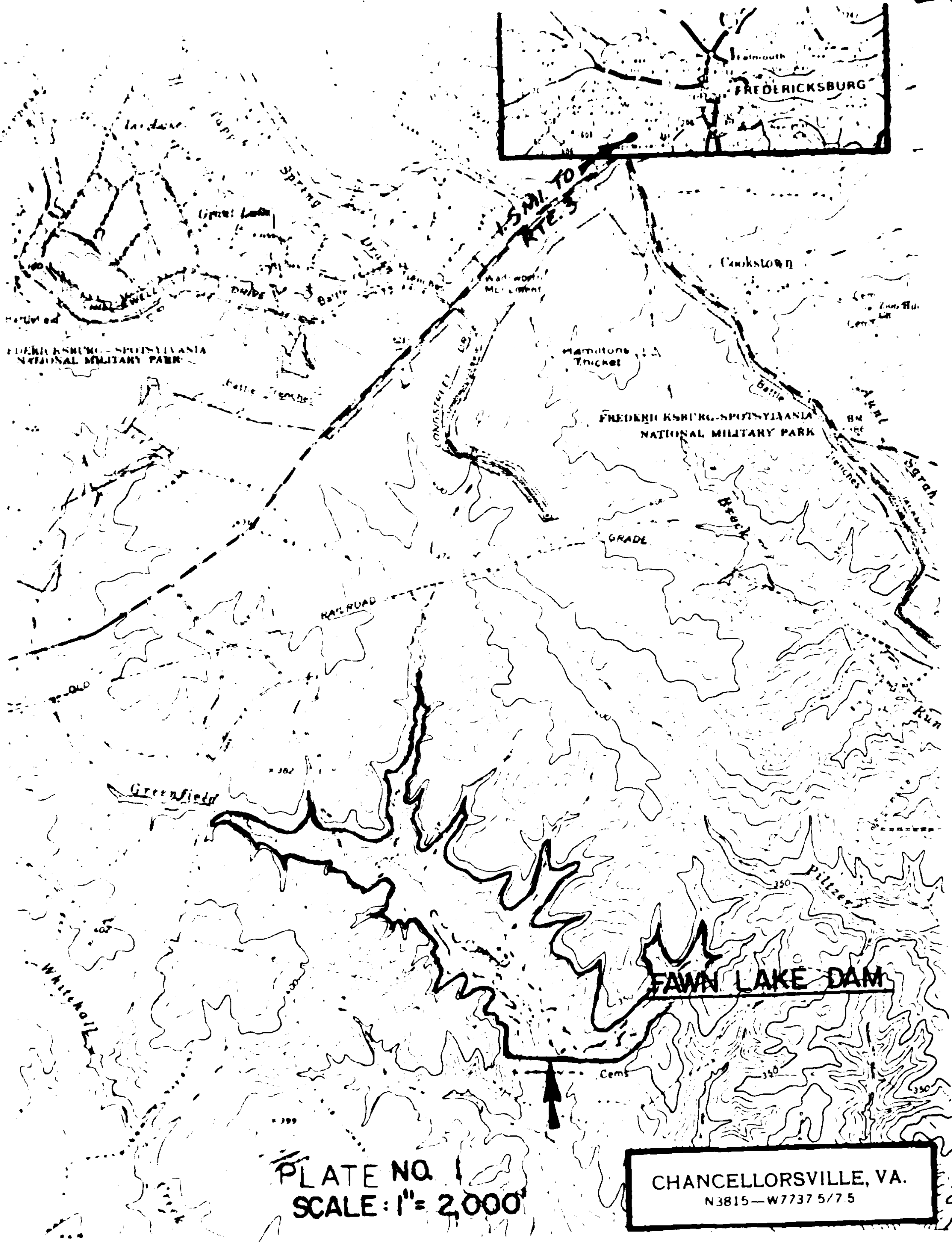
7.3.2 The emergency spillway should be reseeded in order to establish a better stand of vegetative cover. This will minimize future surface erosion.

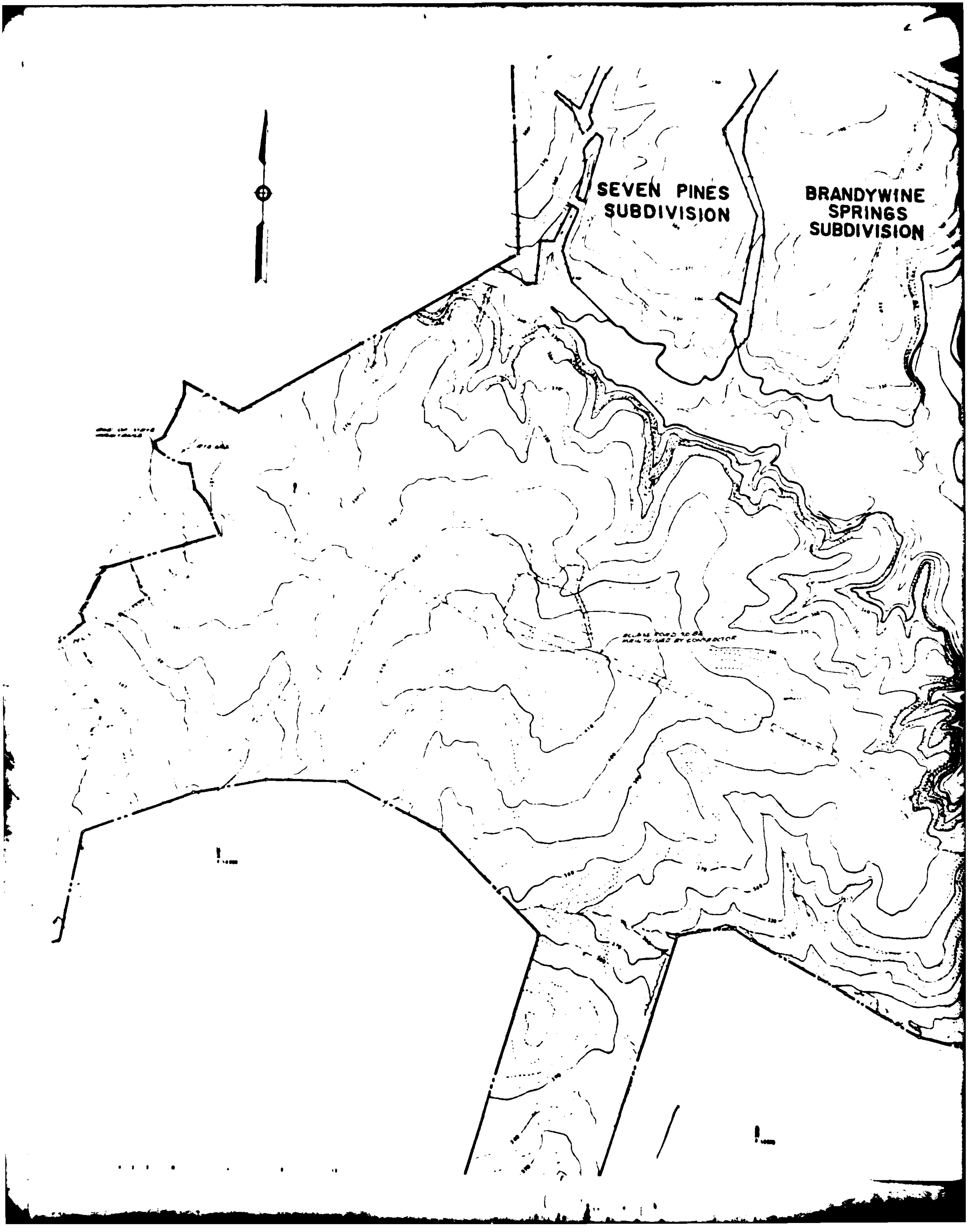
7.3.3 Eroded areas or washes along the right downstream slope-abutment contact and also along the toe of the downstream slope should be examined during the normal maintenance schedule. Increased erosion may require local grading and reseeding.

7.3.4 Seepage observed along the right downstream toe should be monitored quarterly for increased flows and turbidity. It these conditions should occur, a Professional Engineer with an expertise in Geotechnical Engineering should be contacted to evaluate the problem and make recommendations for required corrective measures. The importance of this precaution cannot be overemphasized since seepage occurring along the former stream channel poses a potential for piping, and thus embankment failure. Seepage observed along the left downstream toe appears to be related to spring seepage, however, a periodic examination of this area is suggested to insure that migration of saturation and erosion of the downstream toe does not occur here.

# APPENDIX I

# MAPS AND DRAWINGS


FREDERICKSBURG
1.5 MI. TO RTE. 3
Cookstown
FREDERICKSBURG-SPOTSYLVANIA NATIONAL MILITARY PARK
Hamiltons Thicket
FREDERICKSBURG-SPOTSYLVANIA NATIONAL MILITARY PARK
Battle Trenches
GRADE
RAILROAD
OLD
Greenfield
Pittzer
Run
Whitehall
FAWN LAKE DAM
Cems
PLATE NO. 1
SCALE: 1" = 2,000'
CHANCELLORSVILLE, VA.
N3815—W7737.5/7.5


SEVEN PINES
SUBDIVISION
BRANDYWINE
SPRINGS
SUBDIVISION

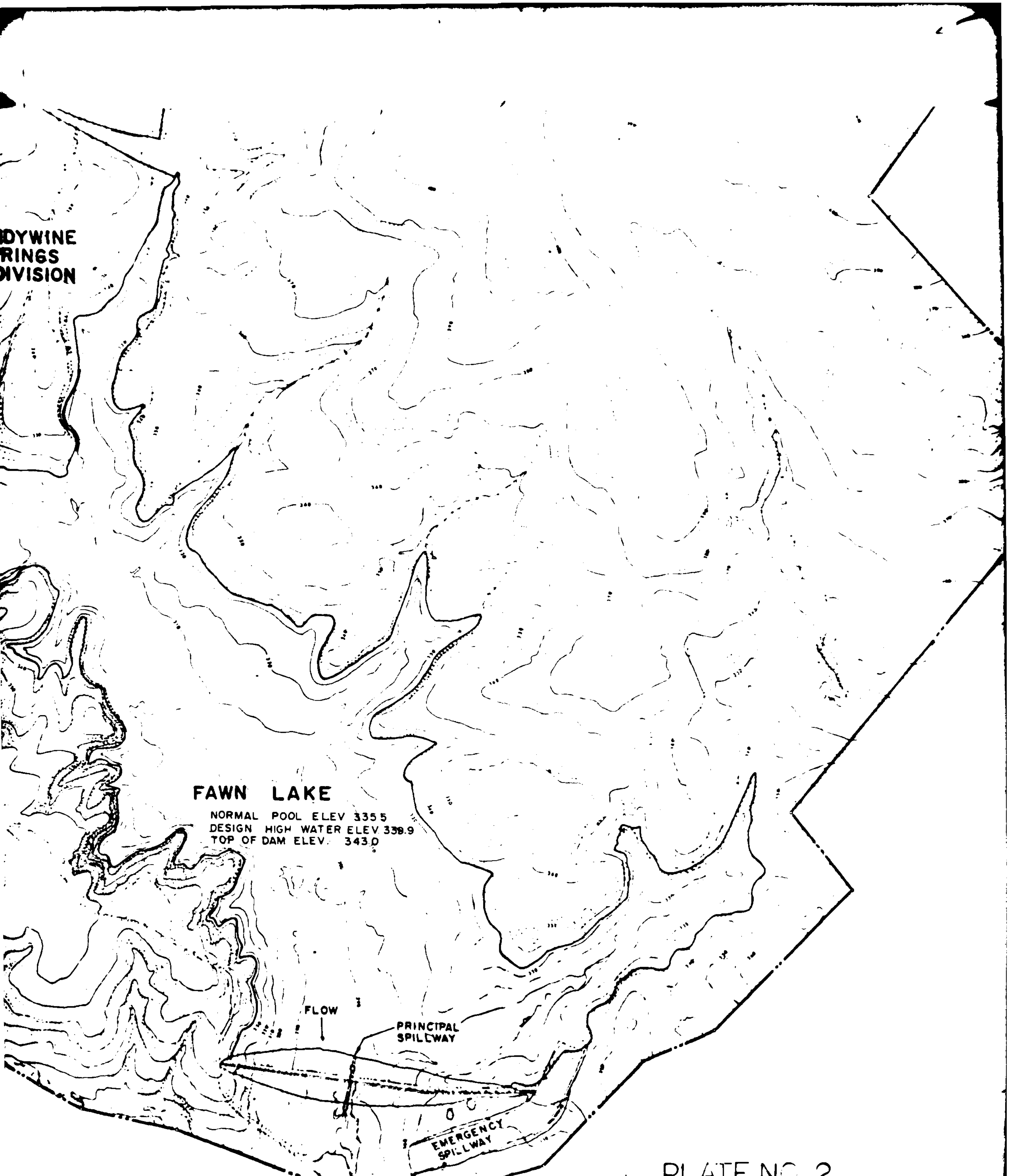
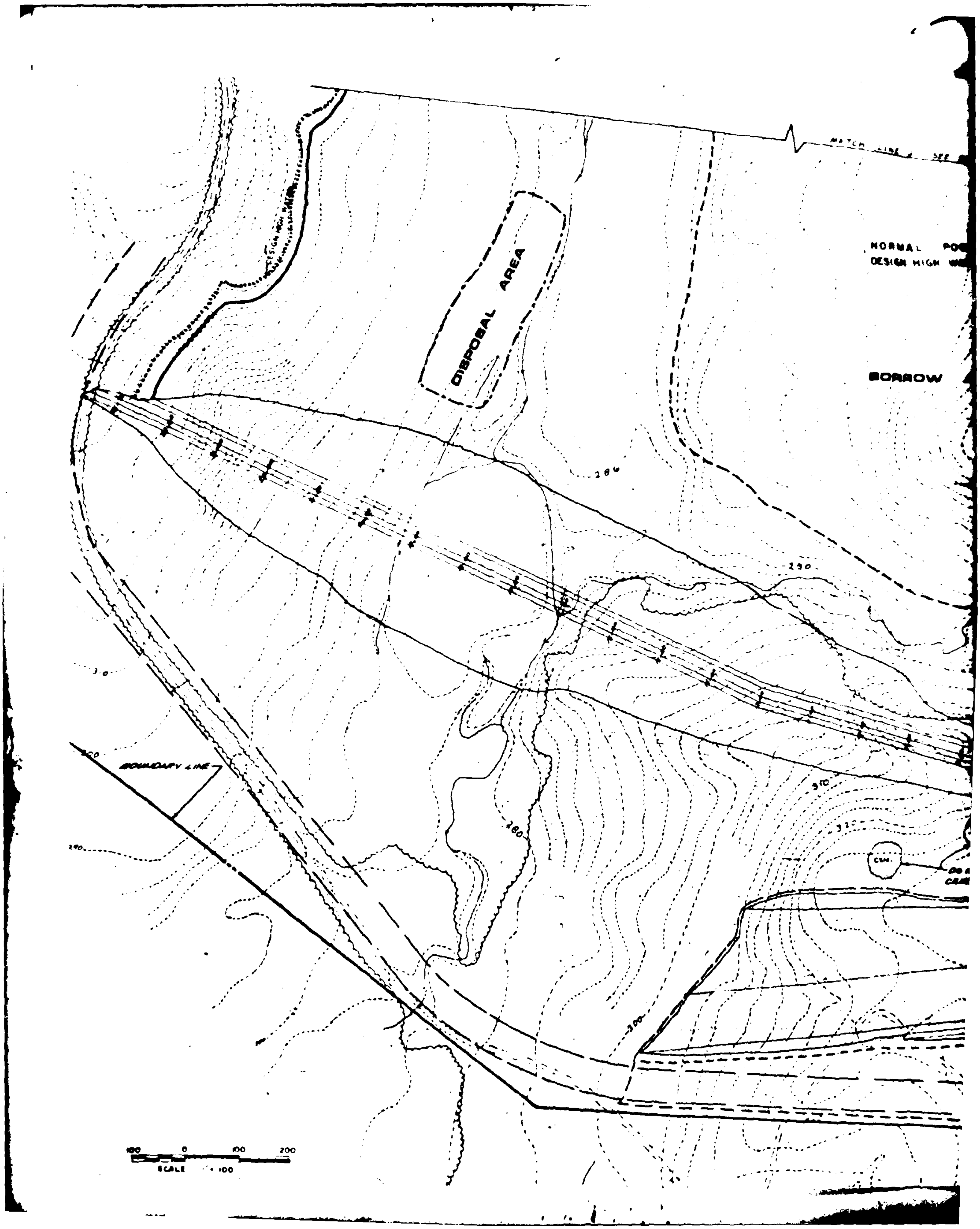



PLATE NO. 2

SITE LOCATION MAP

FAWN LAKE COMMUNITY

AMERICAN CENTRAL CORPORATION


DISPOSAL AREA
BORROW
NORMAL PO
DESIGN HIGH W
DESIGN HIGH WATER
BOUNDARY LINE
280
290
300
310
320
SCALE
100 0 100 200

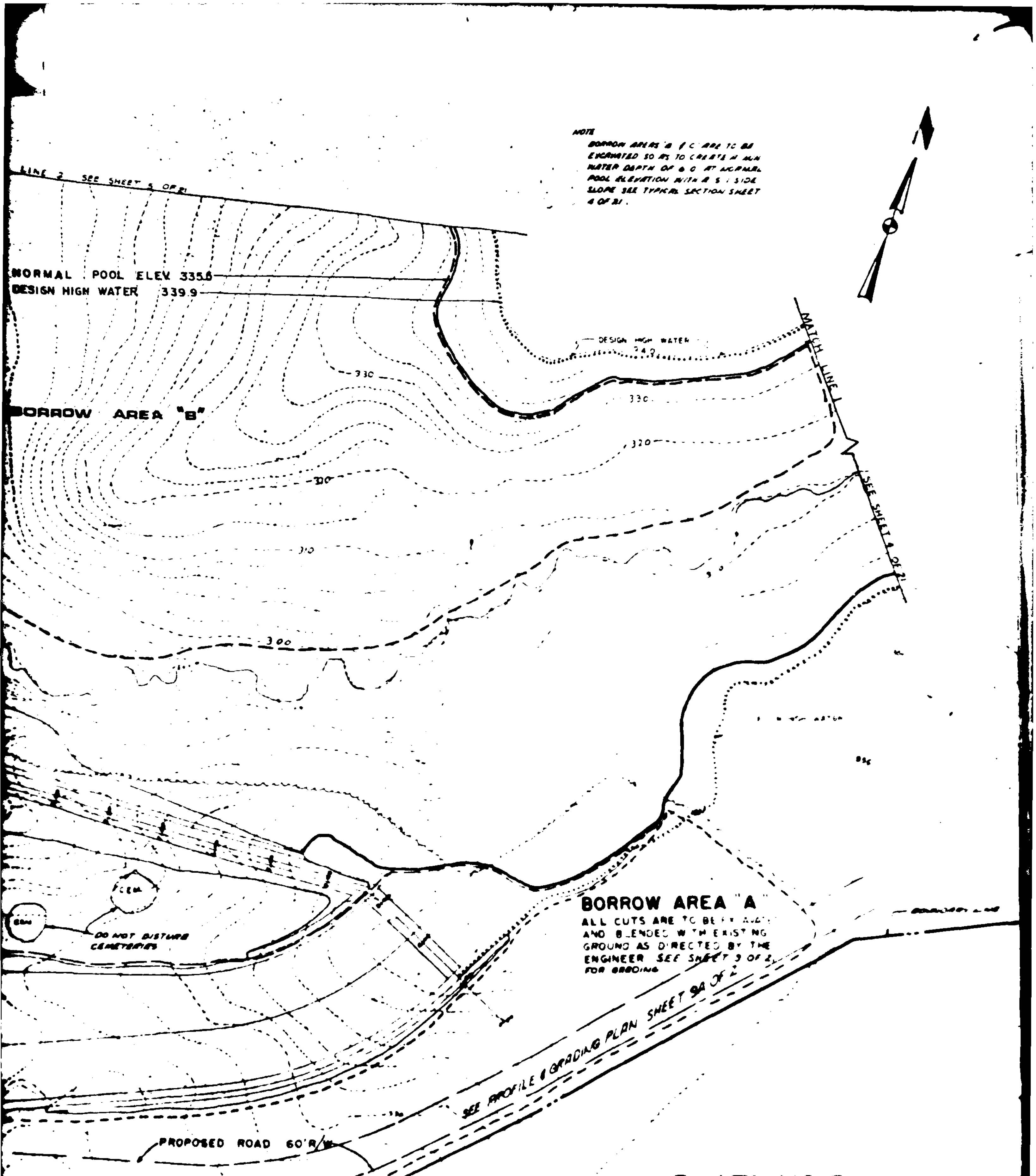
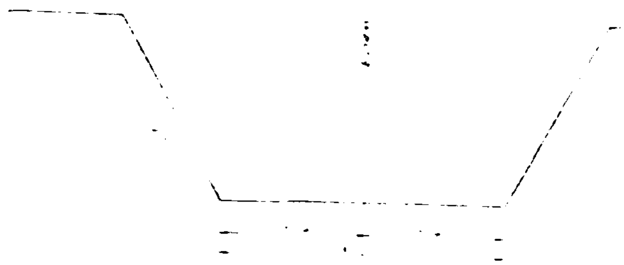



PLATE NO. 3

SITE LOCATION MAP
FAWN LAKE COMMUNITY
AMERICAN CENTRAL CORPORATION

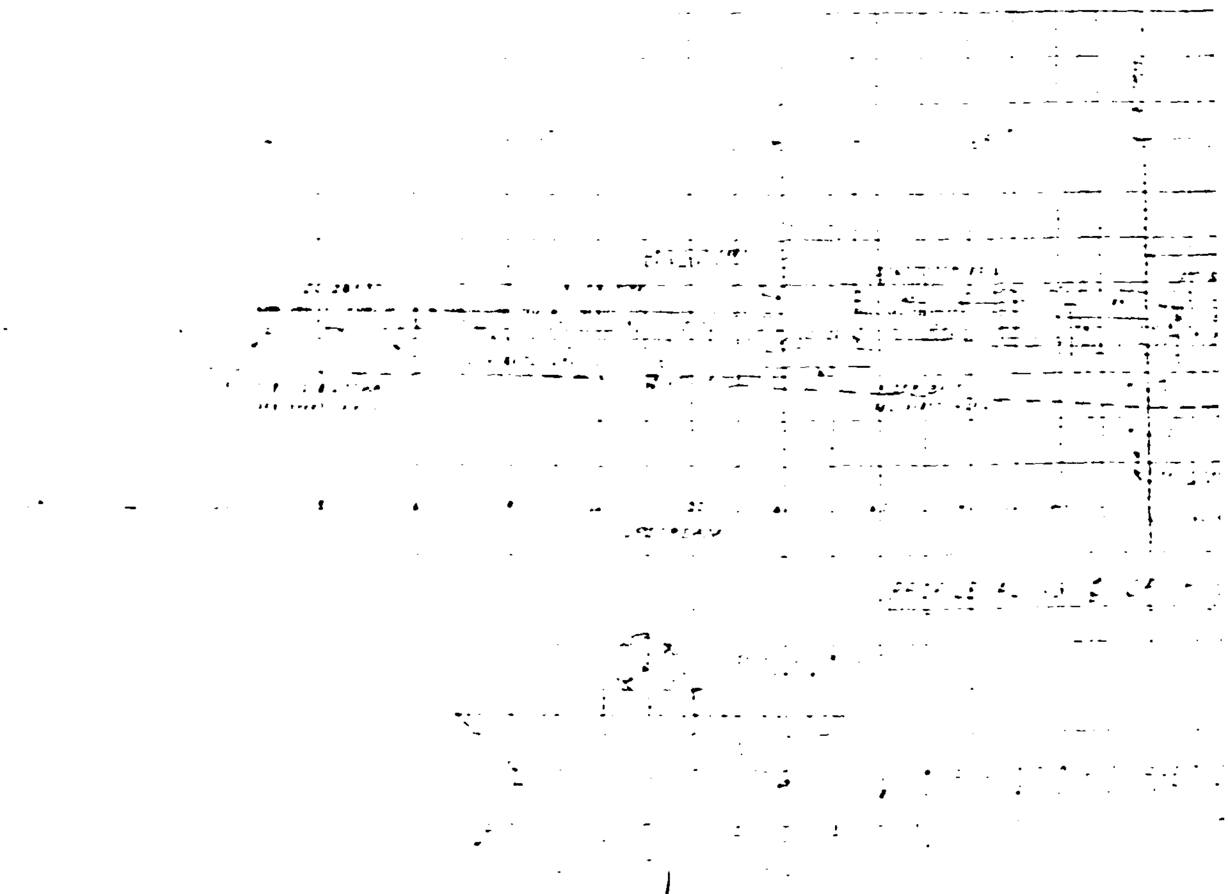


[illegible] ATE NO. [illegible]

FA[illegible]N [illegible]AKE [illegible]TY

[illegible]

[illegible]



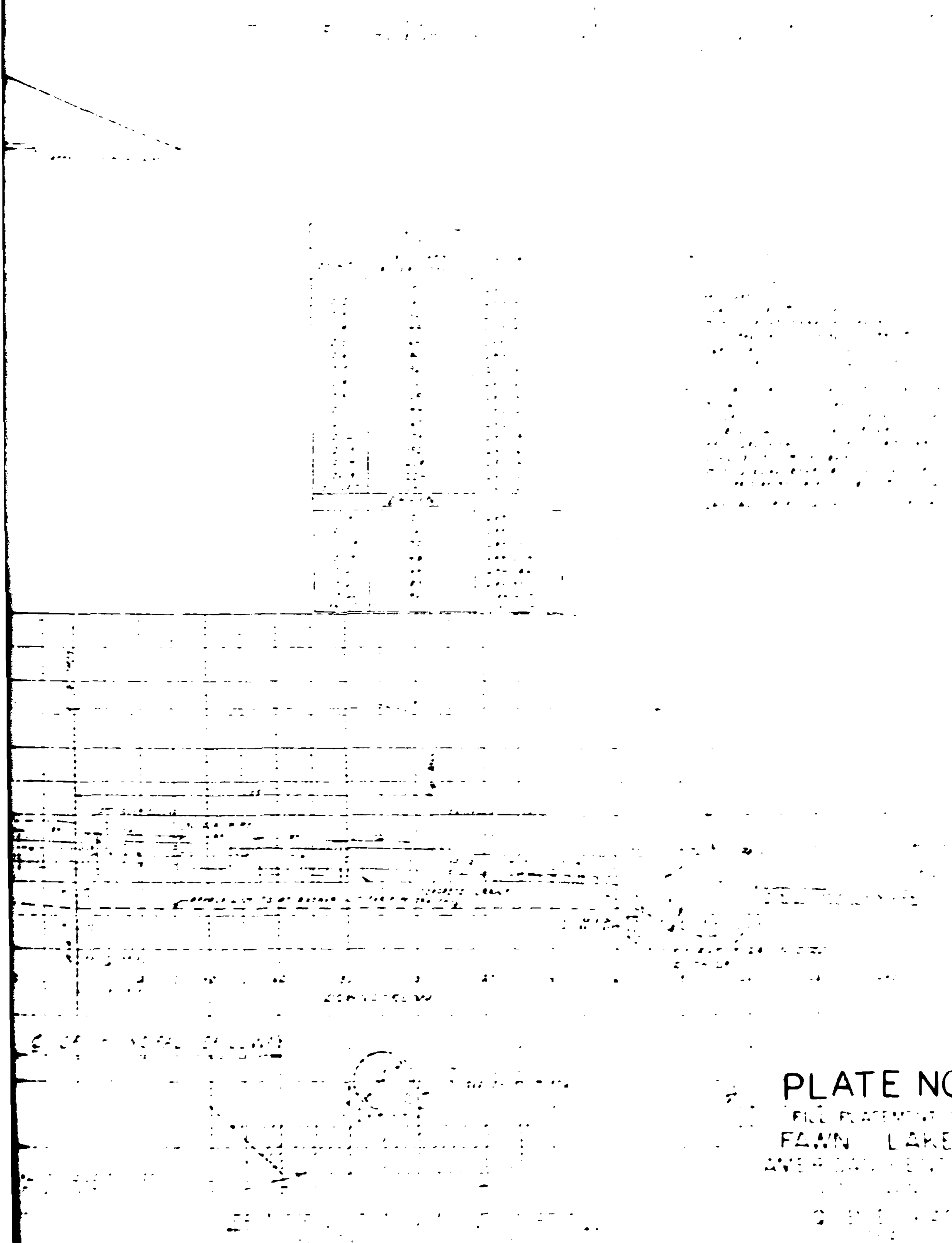


PLATE NO. 5

FAWN LAKE

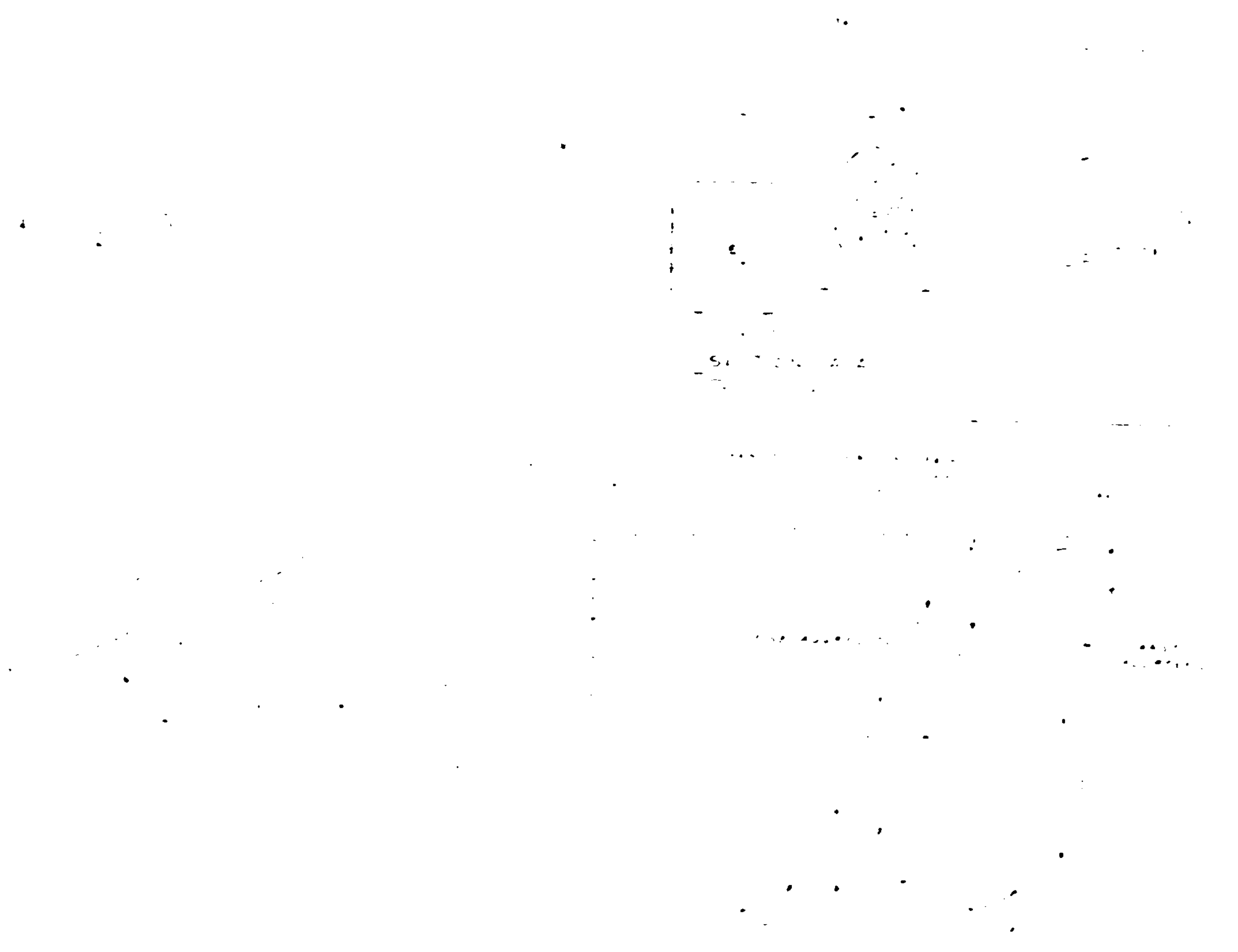
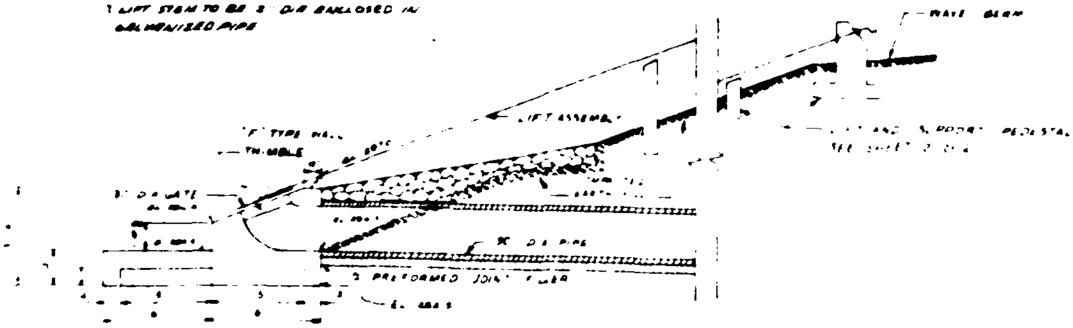


PLATE NO. 6

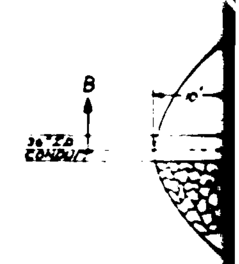
## HEADWALL & SCOUR APRON

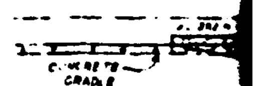
WATER CONTROL GATE (30" DIA)

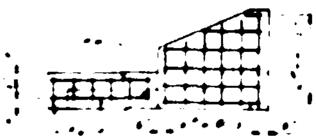
1. 30" x 30" HEAVY DUTY SLUICE GATE WITH ROUND OPENING FOR A SEATING HEAD OF [illegible] AND AN UNSEATING HEAD OF [illegible]
2. "F" TYPE WALL THIMBLE 10" DEEP WITH A ROUND OPENING
3. STEM GUIDES, ANCHOR BOLTS AND RISING STEM ARE TO BE SPACED ACCORDING TO MANUFACTURERS RECOMMENDATIONS
4. CRANK OPERATED LIFT AND PEDESTAL BASE AS RECOMMENDED BY MANUFACTURER
5. GATE SHALL BE INSTALLED ACCORDING TO MANUFACTURERS RECOMMENDATIONS
6. SHOP DRAWINGS ARE TO BE APPROVED BY THE ENGINEER
7. LIFT STEM TO BE 2" DIA ENCLOSED IN GALVANIZED PIPE




## SECTION B-B
## HEADWALL & SCOUR APRON



## DETAIL

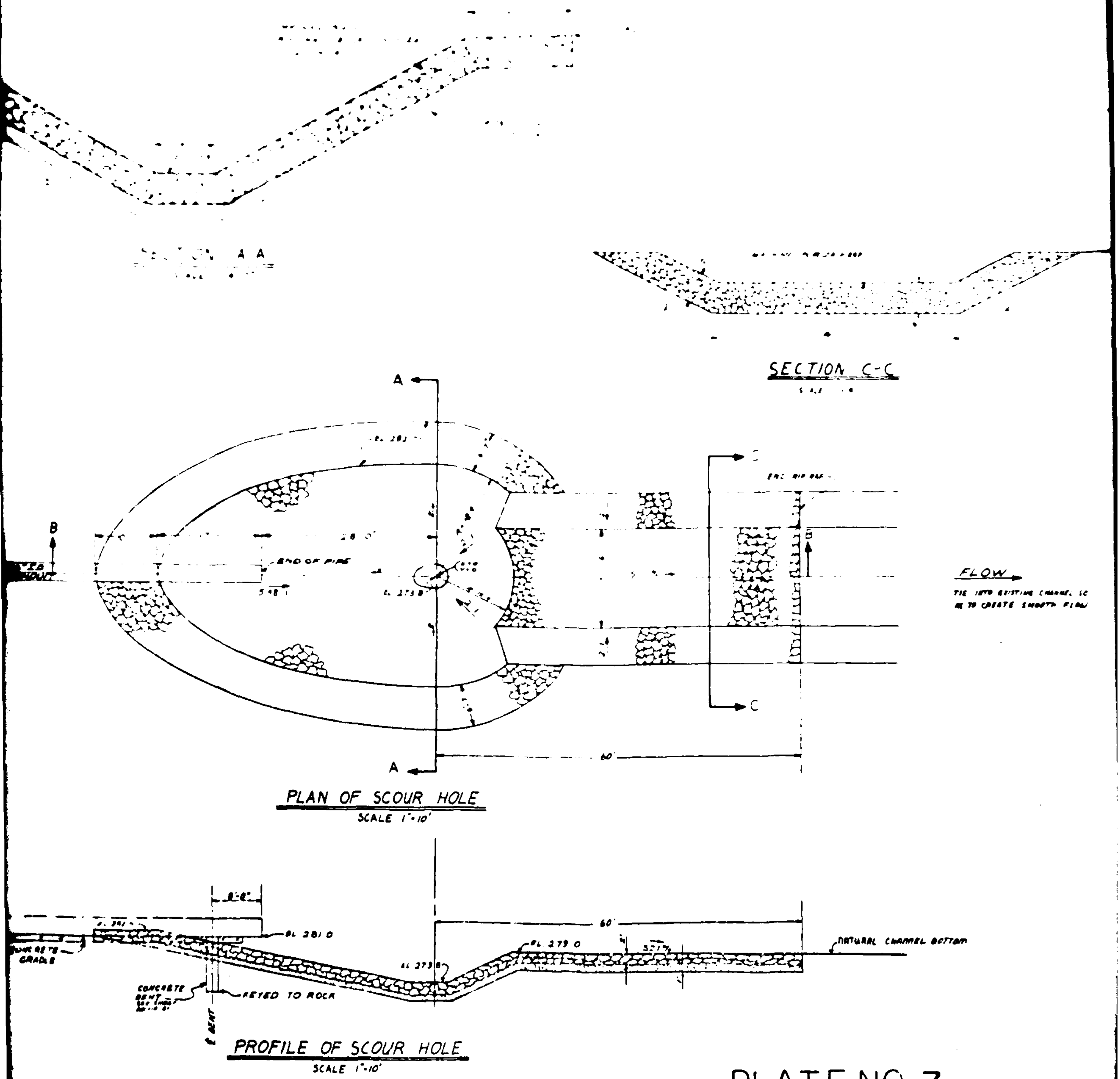



PLATE NO. 7

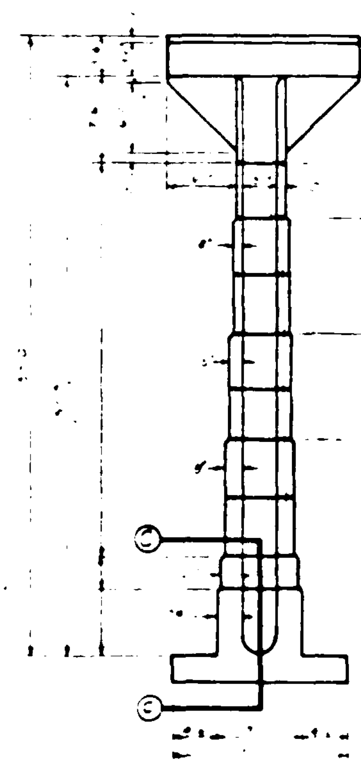
DETAILS-SCOUR HOLE, HEADWALL, GATE & WEIR

**FAWN LAKE COMMUNITY**

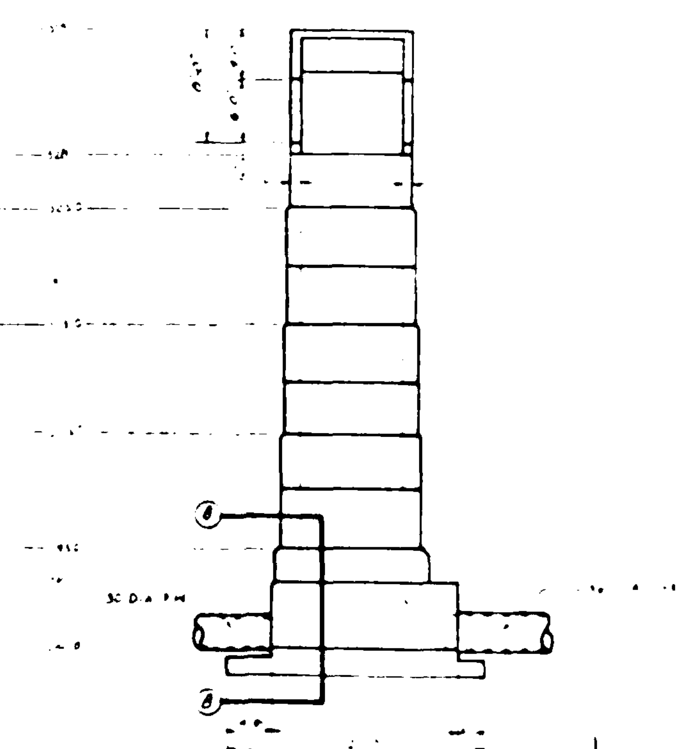
AMERICAN CENTRAL CORPORATION

TOP PLAN

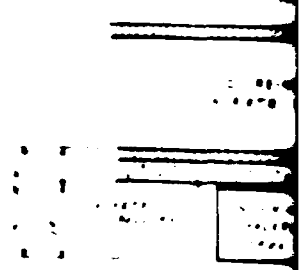
TYP PLATE CONSTRUCTION JOINT

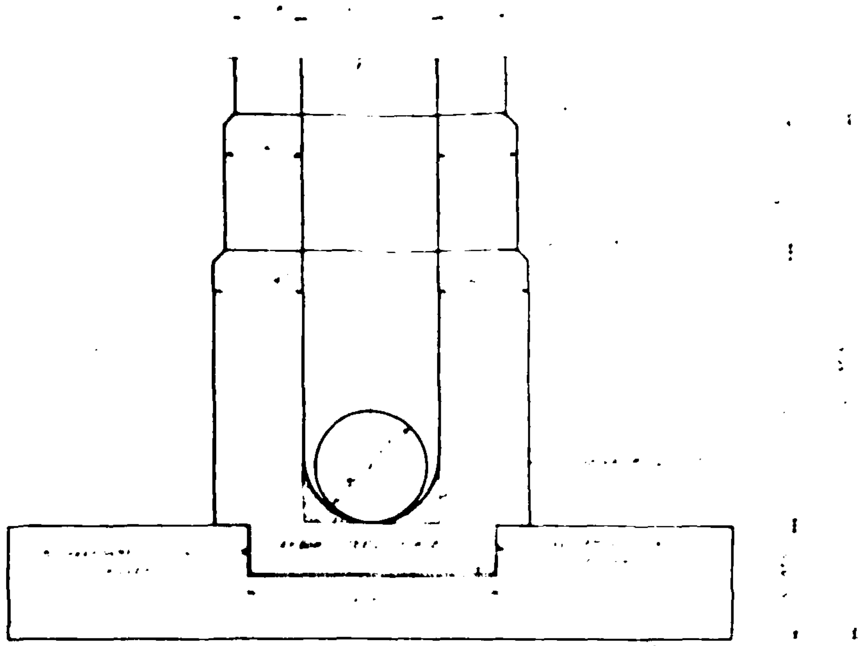


SECTION A A

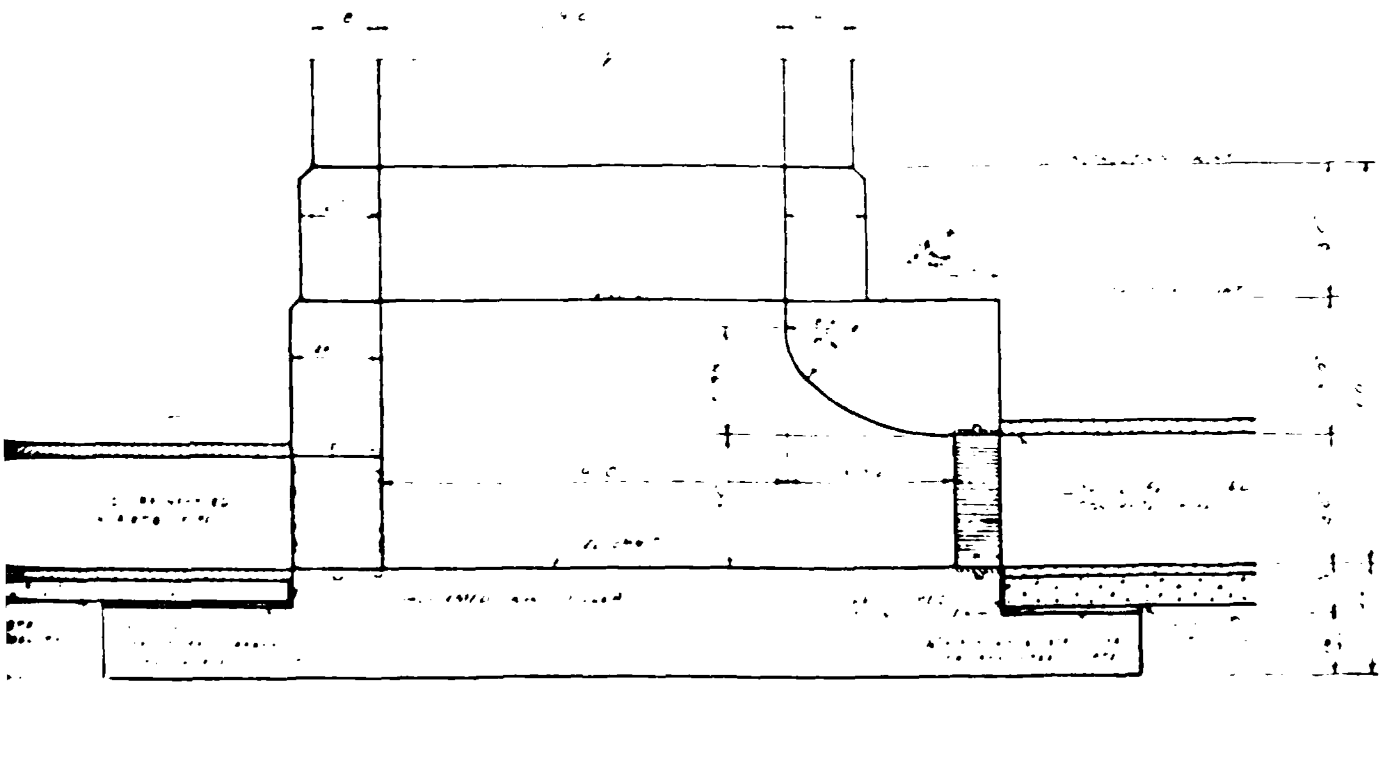


SIDEWALL ELEVATION





SECTION B-B

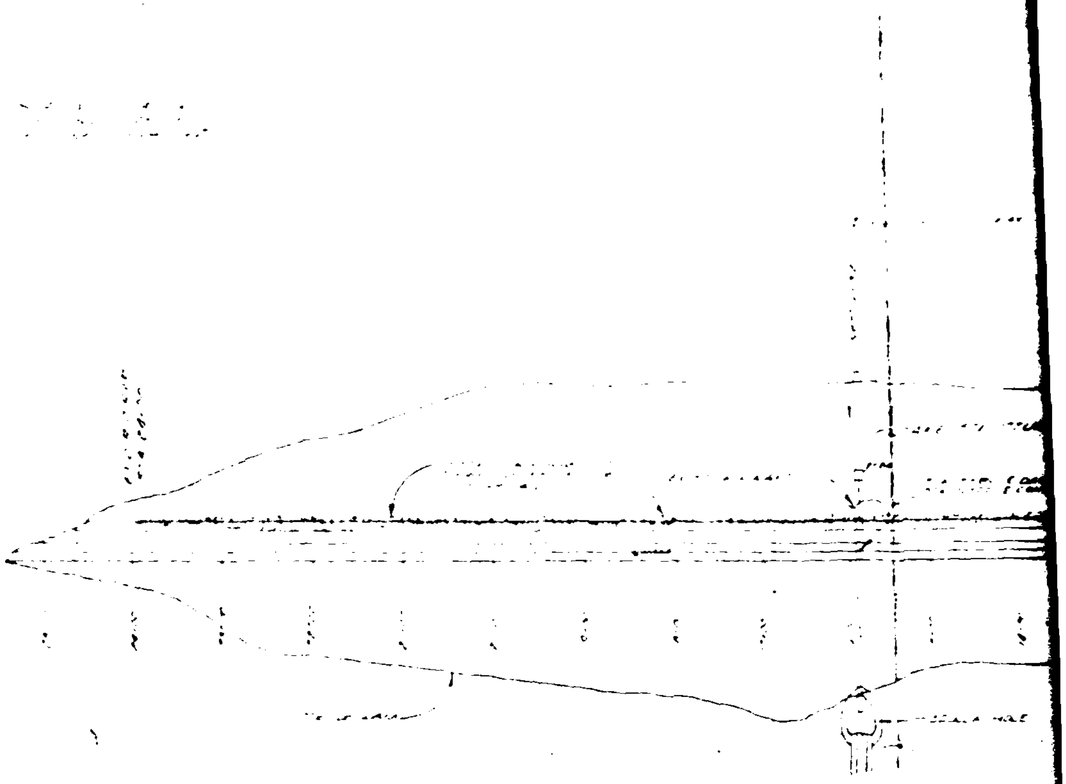


SECTION C-C

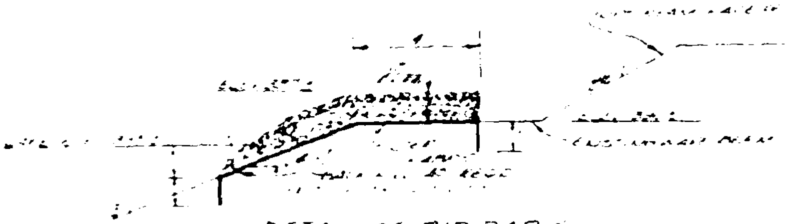
PLATE NO. 8

RISER DETAILS

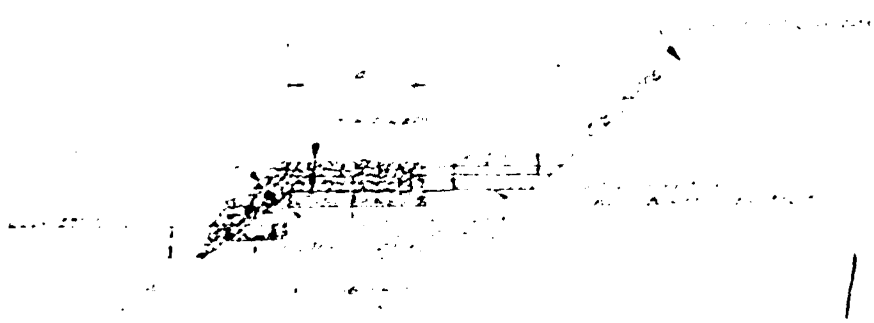
FAWN LAKE COMMUNITY

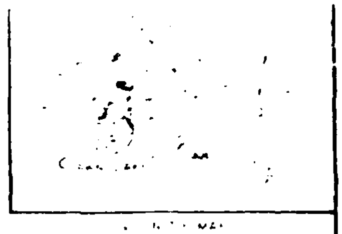


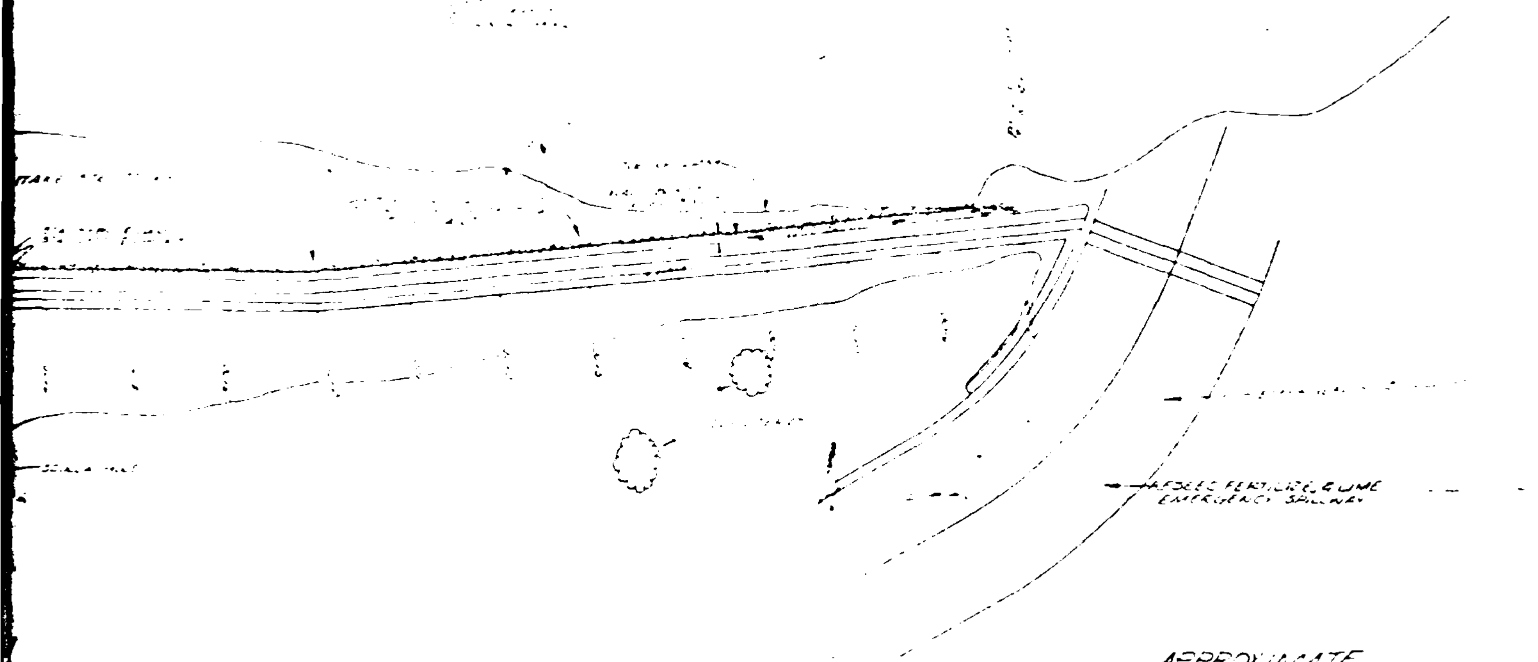
PLAN



DETAIL OF RIP-RAP &
ALTERNATE FILTER FABRIC

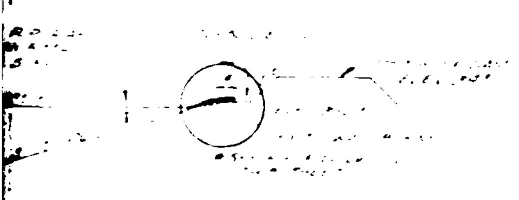







APPROXIMATE
QUANTITIES

| | |
|---|---|
| 6" FILTER BLANKET | 900 TONS |
| 12" CLASS I RIP RAP | 1800 TONS |
| EARTHWORK | 190 C Y |
| TOPSOIL & SEED | 1340 S Y |
| RESEED, FERT. & LIME | 6 AC |
| ALTERNATE FILTER FABRIC | 2533 S Y |



TYPICAL SECTION

2

PLATE NO. 9

J. K. TIMMONS & ASSOCIATES INC
CONSULTING ENGINEERS
1314 W MAIN ST RICHMOND VA

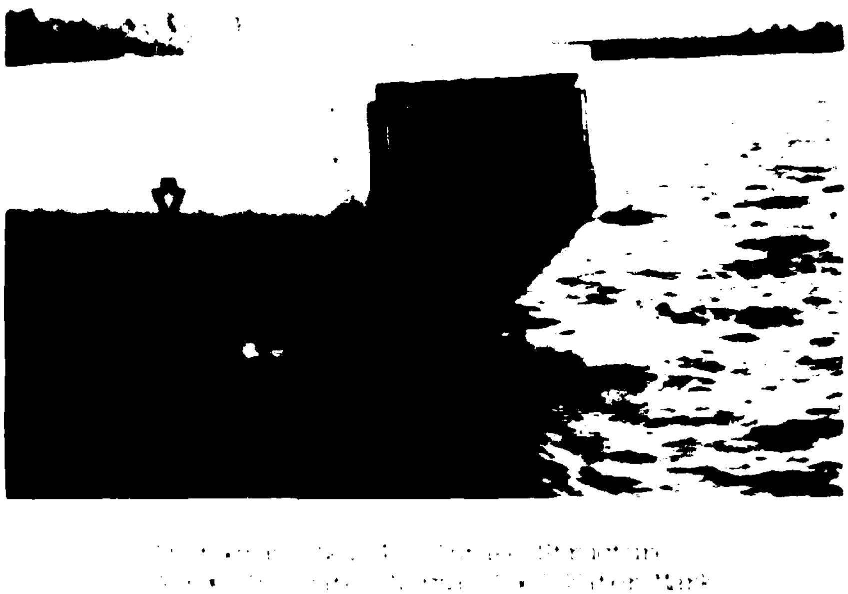
FAWN LAKE DAM
SHORE LINE PROTECTION
SPOTSYLVANIA CO., VIRGINIA

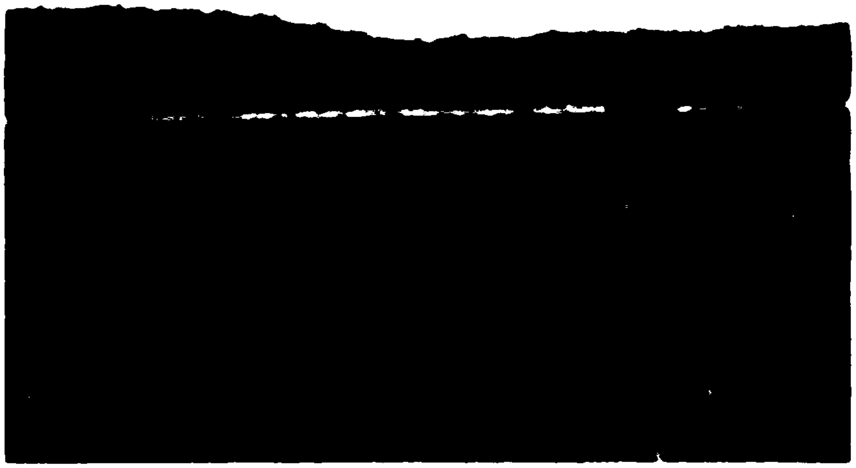
[illegible]

[illegible]

Photograph No. 1 - [illegible]

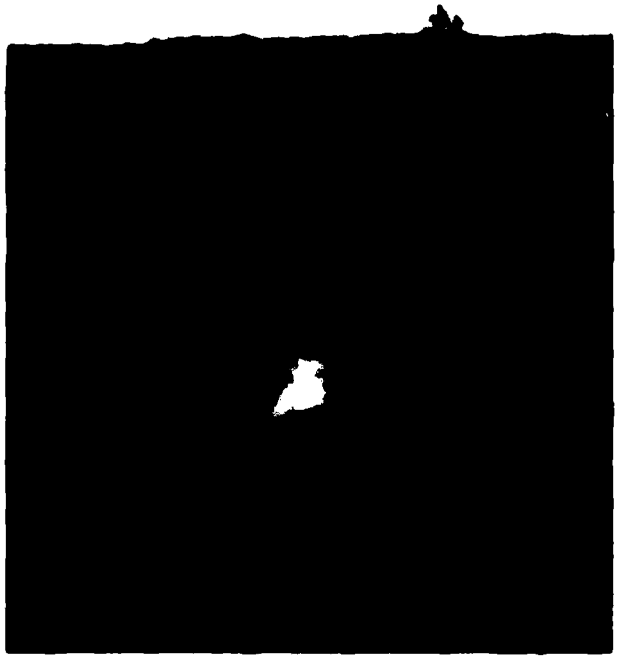
Photograph No. 2 - Downstream Slope
Note Cattails in Old Streambed (Arrow)





[illegible]

[illegible]



Photograph No. 7 - Plunge Pool
and Outlet Channel

# APPENDIX III

# FIELD OBSERVATIONS

Check List
Visual Inspection
Phase I

Name Dam Fawn Lake Dam County Spotsyl State Virginia Coordinators $38^{\circ}$ 15.5' Lat / $77^{\circ}$ 42.9' Long

Date(s) Inspection April 20, 1981 Weather Cloudy, Windy Temperature $55^{\circ}$ F

Pool Elevation at Time of Inspection El 332 msl Tailwater at Time of Inspection 280 msl

Inspection Personnel:

Schnabel Engineering Associates, P.C.
Stephen G. Werner
Gilbert T. Seese

J. K. Timmons and Associates, Inc.
Robert G. Roop, P.E.
Steve Oddi

State Water Control Board
Hugh Gildea, P.E.

Owner Representative:
Edgar Webb

Stephen G. Werner, Recorders
G. T. Seese

EMBANKMENT

| [illegible] | OBSERVATIONS | REMARKS OR RECOMMENDATIONS |
|---|---|---|
| [illegible] | The slopes, crest and abutment contacts were inspected and no cracks were noted. | None |
| [illegible] | No unusual movements or cracking were noted on the dam or downstream beyond the embankment toe. | None |
| [illegible] OF [illegible] | Several eroded areas along upstream face due to wave action and lack of vegetation. On the downstream face to the east of the normal spillway outlet, two [illegible] eroded areas [illegible] wide and [illegible] These two areas were [illegible]. Slopes of [illegible] were [illegible] of the upstream and downstream slopes. | Wave action is causing some erosion below the riprap. Lake level is [illegible] below normal. |
| [illegible] | The vertical and horizontal alignment of the dam crest [illegible] good. | None |
| [illegible] | [illegible] observed along the upstream face and in the [illegible] The riprap appears to be [illegible] | Riprap was placed on the upstream slope in [illegible] |

EMBANKMENT

| VISUAL EXAMINATION OF | OBSERVATIONS | REMARKS OR RECOMMENDATIONS |
|---|---|---|
| JUNCTION OF EMBANKMENT AND ABUTMENT, SPILLWAY AND DAM | Scattered erosion approximately 1 ft wide and 1 to 2 ft deep was noted along the right downstream abutment. Otherwise, conditions were good at the contacts. | The erosion appears to be caused by runoff. |
| ANY NOTICEABLE SEEPAGE | Wet areas were located along the left downstream toe from the principal spillway outlet [illegible] for a distance of approximately 100 ft, cattails and [illegible] exist in this area. The water was clear and little or no [illegible] was present. See the field sketches for approximate locations and dimensions of wet areas. [illegible] large wet area containing marsh grass and dead trees was observed just beyond the right downstream toe. No flow was observed at either location. | During an earlier [illegible] it was reported that [illegible] were known to exist [illegible] left side. The [illegible] channel was said [illegible] along the right side. |
| DRAINS | 6 inch toe drain outlet is located on the right side of the principal spillway outlet pipe. Water coming out of the CMP pipe was clear and flowing between 2 to 4 GPM±. | The drain appears to be functioning properly. |
| MATERIALS | Surface soils on the embankment ranged from red to reddish brown fine sandy silty clay (CL) to fine to medium clayey sand (SC). The embankment ties into residual or colluvial soils in the abutment areas consisting of sand, silt, and clay mixtures with rock fragments. Overall, the materials consist basically of red silty clay to clayey silt, trace fine sand (CL to MH). | - |
| VEGETATION | Slopes were grass covered except for a few eroded areas on the downstream slope (see Field Sketch No. 1) and along the water line where some erosion due to wave action was observed. | - |

PRINCIPAL SPILLWAY

| [illegible] | OBSERVATIONS | REMARKS AND RECOMMENDATIONS |
|---|---|---|
| [illegible] | Concrete overflow intake structure with bar screens | In good condition |
| [illegible] | None | - |
| [illegible] | One 36 inch reinforced concrete pipe. Plunge pool and riprap [illegible] good condition. Channel is 20 ft wide and has [illegible] side slopes | In good condition. Some vegetation growing in channel due to low flow. |
| [illegible] | [illegible] ft across top of dam. | - |
| [illegible] | Reported to be operational by Mr. Edgar Webb, Caretaker | The gate has been operated several times in the last 12 months due to construction and riprap placement. |
| [illegible] | None | - |

| | [illegible] | REMARKS OR RECOMMENDATIONS |
|---|---|---|
| [illegible] | [illegible] of a 200 ft wide [illegible] with [illegible] side slopes | Vegetative cover is sparse. Needs reseeding. |
| [illegible] | [illegible] with 200 ft wide bottom and [illegible] side slopes. | Needs reseeding. |
| [illegible] | [illegible] with 200 ft wide bottom and [illegible] side slopes. [illegible] is sparse. Some minor [illegible] | Needs reseeding. |
| [illegible] | None | - |
| [illegible] | None | - |

## INSTRUMENTATION

| [illegible] | OBSERVATIONS | REMARKS OR RECOMMENDATION |
|---|---|---|
| [illegible] | None observed | - |
| [illegible] | None observed | - |
| [illegible] | None observed | - |
| [illegible] | None observed | - |
| [illegible] | None observed | Should be installed. |
| [illegible] | None | - |

| VISUAL EXAMINATION | OBSERVATIONS | REMARKS AND RECOMMENDATIONS |
|---|---|---|
| SLOPES | Side slopes are approximately 4H:1V. Slopes on the east side are grassed to heavily wooded and are gentle to moderately steep. Slopes on the west side are moderately steep and heavily wooded. Slopes appeared stable. Some minor erosion was observed around the lake bank due to low lake level. The water level is 2 ft $\pm$ below the riprap. The reservoir is free of debris. | Erosion will cease once return to normal lake level. |
| SEDIMENTATION | None observed. | |

DOWNSTREAM CHANNEL

| [illegible] EXAMINATION OF | OBSERVATIONS | REMARKS OR RECOMMENDATIONS |
|---|---|---|
| [illegible]TION ([illegible]STRUCTIONS, [illegible]RIS, ETC.) | Heavily wooded. Includes a broad flood plain. The straight channel section from the outlet pipe to the old stream channel includes scattered clusters of cattails and marsh grass. Wooded stream after 500 ft. | n = 0.1 over bank<br>n = 0.05 channel |
| [illegible]S | Downstream, moderately steep natural wooded slopes bound the right side of the stream while gentle (10H:1V±) to moderate grassed to wooded slopes bound the left side. The flood plain is approximately 500 ft wide immediately below the dam. No slope failures were noted. The channel is approximately 3 ft deep and 20 ft wide with 2H:1V side slopes. Along either side of the channel is a broad floodplain with gentle slopes. It is heavily wooded beyond the intersection with the emergency spillway and discharge channel. | |
| [illegible]XIMATE NO. [illegible] AND [illegible] | Two dwellings are present 4 miles ± downstream, one at Lake Wright 5 ft± above the lake and another along Route 649 15 ft± above the streambed. | The dwelling at Lake Wright has strong potential for flooding. |

CHECK LIST

ENGINEERING DATA

DESIGN, CONSTRUCTION, OPERATION

| ITEM | | REMARKS |
|---|---|---|
| ...IAL VICINITY MAP | Chancellorville, Virginia USGS 7.5 minute Quadrangle | Included in report. |
| ...SIGN/CONSTRUCTION HISTORY | Design: Quible and Associates<br>Chase City, Virginia<br>Construction: Bishop and Settle Construction Company<br>Alberta, Virginia | Design report available from owner, included in report. Construction records incomplete. Not provided by the owner and not included in report. |
| ... OF DAM | Available for inspection | Included in report. |
| ...CAL SECTIONS OF DAM | Available for inspection | Included in report |
| ... - PLAN<br>DETAILS<br>CONSTRAINTS<br>DISCHARGE RATINGS | Principal spillway, scour hole, headwall, gate, weir and riser detail drawings available for inspection. | Included in report |
| ...WAY- PLAN<br>SECTION<br>DETAILS | Principal spillway - plan, profile, emergency spillway - plan, typical cross sections of diversion channels | Included in report. |
| ... EQUIPMENT - PLAN<br>DETAILS | Available for inspection | Included in report. |

| | | |
|---|---|---|
| [illegible] | [illegible] | REMARKS |
| [illegible] | [illegible] | - |
| [illegible] | [illegible] | - |
| [illegible] | The geologic map of Virginia indicates underlying bedrock consisting of metamorphosed sedimentary rock and possible intrusions of younger igneous rock. | - |
| [illegible] | On-site borrow sources described in Geotechnical Engineering Report. | - |
| [illegible] INVESTIGATIONS [illegible] [illegible]-FIELD TEST DATA | Soil test boring logs and laboratory testing results included in Geotechnical Engineering Report. Available for inspection. | Included in report. |
| [illegible]/HYDRAULIC DATA | Available design data | - |

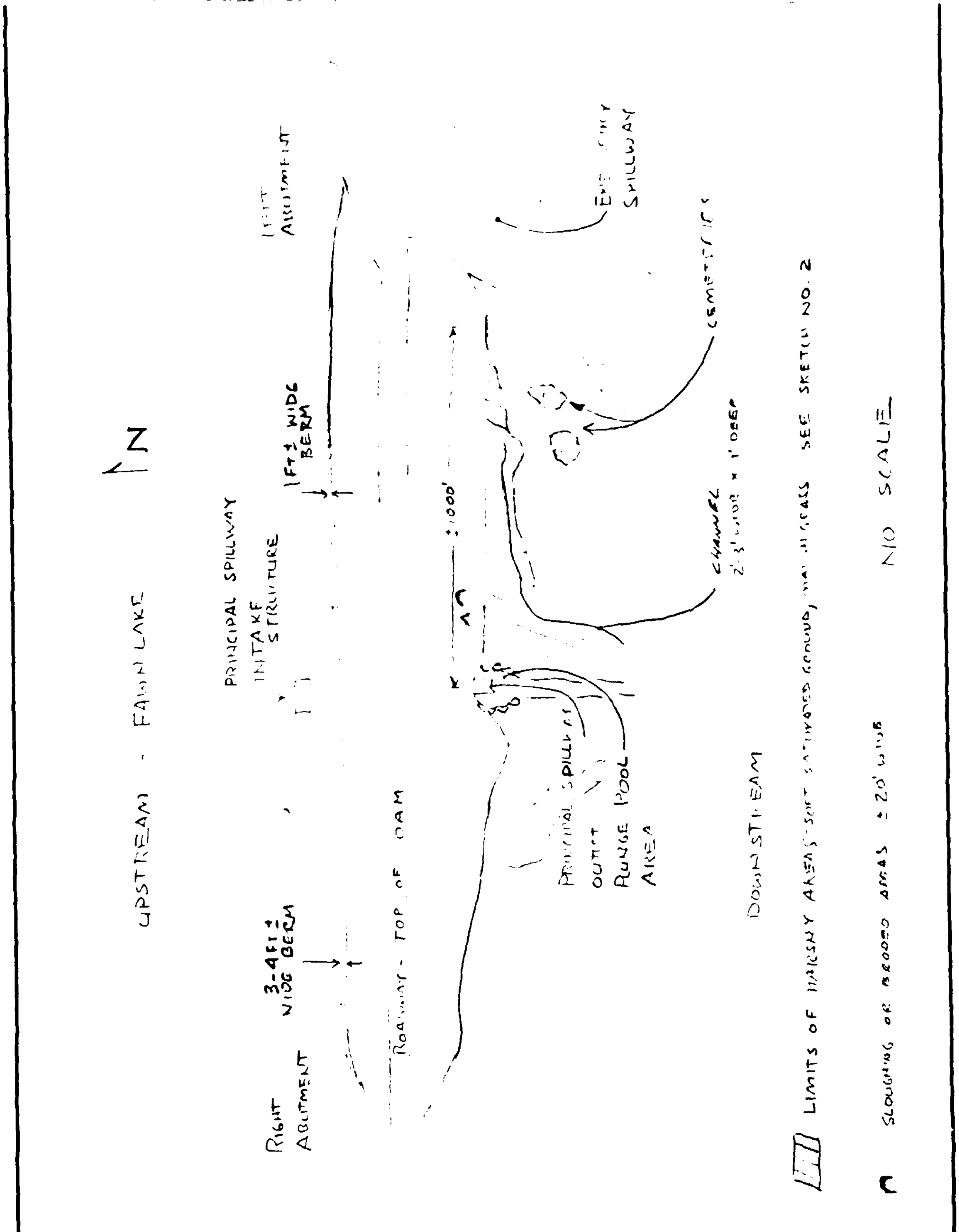
| ITEM | | REMARKS |
|---|---|---|
| [illegible] | Subsurface investigation done by Gaedinger Baker Hampton and Associates, Washington, D. C., dated June 2, 1973. Available for inspection. | Included in report. |
| [illegible] COMPUTATIONS [illegible] HYDRAULICS DAM [illegible] STUDIES | Results of design studies in report by the Geotechnical Engineer. Include hydrology and hydraulics, dam stability and seepage studies. Available for inspection. | Included in report. |
| [illegible] STUDIES [illegible] SURVEYS | Phase I dam inspection made for the owner on June 5, 1979 by Schnabel Engineering Associates, P.C. and J. K. Timmons and Associates, Inc. | Report available from the owner |
| [illegible] | Design and construction of riprap on upstream slope in summer of 1980. | Design data available from the owner. |
| [illegible] OR FAILURE DESCRIPTION REPORTS | None experienced | - |
| [illegible] OPERATION RECORDS | None available. | - |

CONSULTING ENGINEERS
SOIL MECHANICS AND FOUNDATIONS

DATE

CONTRACT NO

FAWN LAKE DAM FIELD SKETCH No 1



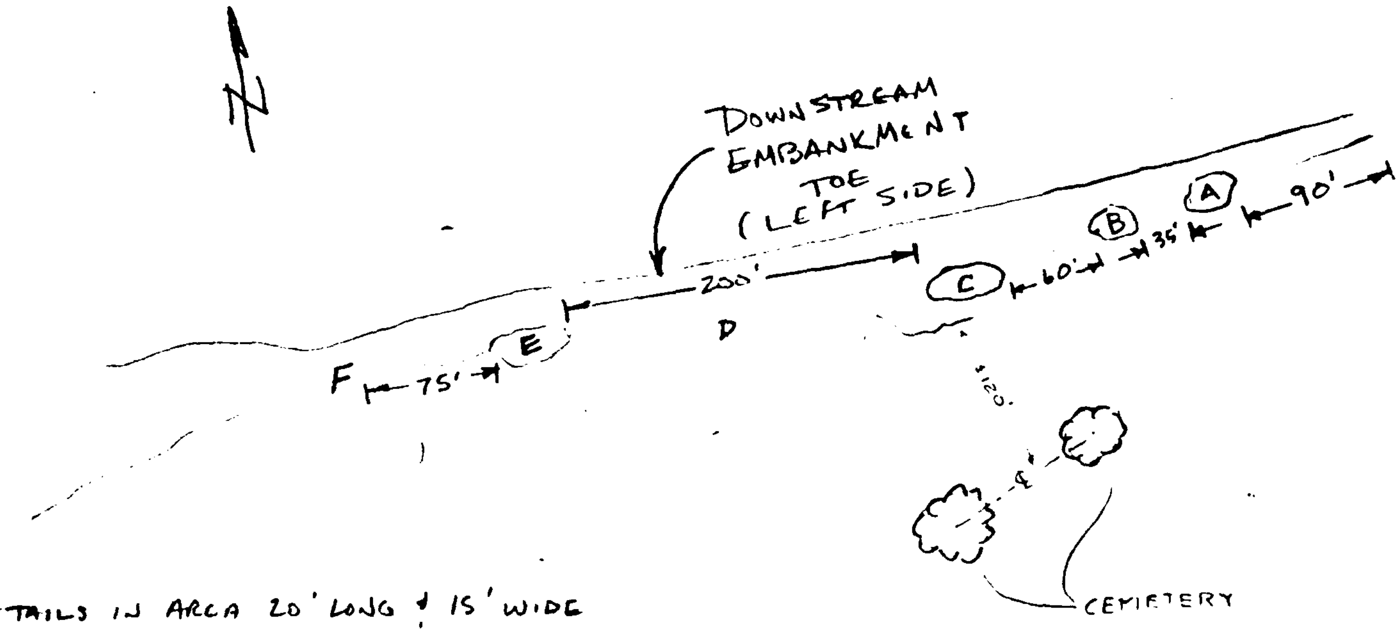
CONSULTING ENGINEERS

SOIL MECHANICS AND FOUNDATIONS

DATE Apr ... '81

CONTRACT NO V31108

## FAWN LAKE DAM FIELD SKETCH No 2




NO SCALE

A. CAT TAILS IN AREA 20' LONG & 15' WIDE

B. CAT TAILS IN AREA 15' LONG & 10' WIDE

C. CAT TAILS IN AREA 40' LONG & 15' WIDE

D. APPROXIMATELY 100' BELOW CEMETERY AN ERODED CHANNEL 2'± DEEP FORMS; FLOW OF SEVERAL GPM

E. CAT TAILS IN AREA 45' LONG & 25' WIDE

F. SURFACE CHANNEL MOVES AWAY FROM EMBANKMENT TOE AT THIS LOCATION; SOME MARSH GRASS PRESENT 150' BELOW PT. F, 300' BELOW PT. F THE CHANNEL IS 15' FROM THE TOE WHERE IT TURNS ABRUPTLY DOWNSTREAM PARALLELING THE COURSE OF THE OUTFALL CHANNEL; 405' TO ℄ OF PRINCIPAL SPILLWAY PIPE

APPENDIX IV

DESIGN REPORT

SUBSURFACE INVESTIGATION

AMERICAN CENTRAL DAM SITE

SPOTSYLVANIA COUNTY, VIRGINIA

JOB NO. 2945

June 2, 1973

Quille & Associates Inc.
P. O. Box 217
Chase City, Virginia

Attention: Mr. Marvin L. Crutchfield

Reference: Subsurface investigation for the proposed American Central Dam site, Spotsylvania County, Va.

GBH Job No. 16181
SCI Job No. 2945

Gentlemen:

The subsurface investigation for the proposed American Central Dam site to be located in Spotsylvania County, Virginia, has been completed. We are submitting, herewith, our engineering report with results of that investigation.

If there are any questions with regard to the information submitted in this report or if we can be of further service to you in any way, please do not hesitate to contact us.

Very truly yours,

GNAEDINGER BAKER HAMPTON & ASSOCIATES

John H. Brewer

John H. Brewer, Ph.D., P.E.
Senior Project Engineer

Delon Hampton

Delon Hampton, Ph.D., P.E.
Chief Engineer

/sr

## INTRODUCTION

The subsurface investigation for the proposed Fawn Lake Dam to be located in Spotsylvania County, Virginia, has been completed. In accordance with your letter of authorization, dated December 28, 1972, seven deep borings along the center line of the dam were performed, and two probe holes along the pipe line and eight auger holes within the borrow areas were performed. The deep borings were completed on April 6, 1973, the auger holes and probe holes were conducted on January 16,1973 and March 29, 1973, respectively. The results of these borings, along with a location diagram, are included in the appendix of this report.

The site inspection revealed the area to be rolling with a maximum difference in elevation of approximately 80 ft. In the vicinity of the proposed dam the land has been used primarily for farming. Vegetation consists of grasses, brushes, and trees. It is our understanding that this site will be developed as a recreational home site.

## SUBSURFACE INVESTIGATION PROCEDURES

The borings were performed with a skid mounted rotary drilling rig which uses various cutting bits and wash water to advance the bore holes. Steel casing was used to keep the bore holes open during the boring operation. and in the pressure testing operations. Representative soil samples were obtained by means of split-barrel sampling procedures in accordance with ASTM Specification D-1586 (see appendix). In the split-barrel sampling procedure, a 2 in. O.D. ( 1 3/8 in. I.D.) split-barrel sampler, 2 ft. long, is driven into the soil by means of a 140 lb. hammer with a free fall of 30 in., and the number of blows of the hammer required to drive the sampler 1 ft.

(after 6 in. of initial penetration) is recorded. This number is termed the "standard penetration resistance" and gives an indication of the relative density of granular soils in situ. In a less reliable manner, it is also indicative of the consistency of cohesive soil and is useful in locating hard strata. All samples were sealed immediately in the field and brought to the laboratory of Soil Consultants, Inc., of Merrifield, Virginia, for further examination, testing, and classification. In conjunction with the drilling along the center line of the proposed dam, Packer tests in accordance with the Earth Manual Designation E-18 of the Bureau of Reclamation 1968 were performed. In the Packer tests, an expandable membrane on the end of an "N" rod is lowered into the hole. Once the Packer is positioned in the bore hole, the membrane is expanded and water is forced through the hole in the center of the rod under pressure. The pressure is measured with a gage, and the volume of water is measured with a flow meter.

In order to investigate the borrow areas and the planned pipe center-lines, hand auger holes and probes were performed, respectively. Representative soil samples of the near surface soils were obtained and studied. All soil samples will be retained at the laboratory of Soil Consultants, Inc., for a period of 60 days after which they will be discarded unless instructions as to their disposition are received.

## TESTING PROGRAM

The testing program consisted of performing the liquid limit, the plastic limit, gradation, compaction, consolidation and triaxial tests. The liquid limit of the soil is the water content at which the soil passes from a plastic state to a liquid state. The plastic limit of a soil is the minimum

[illegible] plastic. The liquid limit and plastic [illegible] in accordance with ASTM Specification [illegible] 424, [illegible]ly. In the gradation tests, both the sieve and [illegible]. The gradation test for soil consists of [illegible]ing the distribution of particle sizes from fine to coarse using a [illegible]ter and a series of sieves. In the consolidation test, a sample of [illegible]lded material compacted near the optimum moisture content was placed into a ring and loaded axially under constant stress. For each loading increment the rate of deformation was recorded. The results of the consolidation tests are used to determine settlements. Two triaxial shear tests were conducted. Representative soil samples from the proposed borrow areas were remolded and compacted near the optimum moisture content and dry density to form a cylinder of soil approximately 3 in. in length and 1.41 in. in diameter. The samples were placed between 2 porous stones and sealed with a rubber membrane. These cylindrical samples were confined by fluid pressure and loaded axially until the soil samples failed. Before the samples were tested, they were saturated, by means of a back pressure, and permeability tests were conducted. All test data are noted in the appendix of the report.

In conjunction with the testing program, all samples were visually classified on the basis of texture and plasticity in accordance with the Unified Soil Classification System. The estimated group symbols according to this system is included in the parentheses following the description of the soil on the boring logs. A brief explanation of the Unified Soil Classification System is included in the appendix of this report.

## SUBSURFACE CONDITIONS

The *in situ* soil conditions encountered in the subsurface investigation of the Fawn Lake Dam will be discussed under the following subheadings: Subsurface conditions along the centerline of the dam, subsurface conditions along the centerline of the overflow pipe, and subsurface conditions in the borrow areas.

Subsurface conditions along the centerline of the dam. The soil profiles, based on logs provided by Soil Consultants, Inc., from the results of the 7 borings performed along the centerline of the dam, vary considerably. The subsurface soils consist generally of clays, silts, and weathered rock. In general, the clay overlies the silt which in turn overlies the weathered rock. Results of boring B-2 is fairly typical of the results of borings B-1, B-3, B-5, and B-6. At boring location B-2 the surface stratum, which consists of grayish brown stiff clay with traces of sand and quartz gravel, is approximately 3 ft. thick. From a depth of approximately 3 ft. to 33 ft. below the existing surface, there occurs a grayish brown tough to very tough silt with traces of sand. From a depth of approximately 33 ft. to approximately 52.2 ft. there occurs a hard brown silt. From 52.5 ft. to 60 ft. a gray chlorite schist was encountered.

The distinguishing feature of boring B-4 is the occurrence of gray sandy shale at a depth of approximately 9 ft. below the existing surface. This shale was cored to a depth of 19.5 ft., at which point the boring was terminated.

The soil profile at boring location B-7 varies somewhat from that at boring location B-2. At boring location B-7 the surface stratum consists of

approximately 28 ft. of red silty clay with traces of gravel. From 28 ft. to 48 ft. below the existing surface there occurs a tough purple silty clay with traces of gravel. From 48 to 70 ft. below the existing surface occurs a tough reddish brown and black clayey silt with a trace of gravel. From 72 ft. to 100 ft. below the existing surface, a tough pink clayey silt with a trace of fine gravel was encountered.

Subsurface conditions along the centerline of the pipe. It is our understanding that the overflow pipe will be laid in the vicinity of boring B-4. Two probe holes PH 1 and PH 2 were conducted perpendicular to the centerline of the dam in the vicinity of boring location B-4. The results of probe hole 1 indicates that the surface stratum consists of yellow weathered rock. From a depth of 5 ft. below the existing surface a split-spoon sample was taken. The blow counts for this sample was 50 blows per .2 ft. At probe hole location PH 2 the surface stratum consists of silty clay with some sand and gravel. From a depth of 2.5 ft. to 5 ft. below the existing surface there occurs silty sand and gravel. At this location, a split-spoon sample was taken at 6 ft. and the blow count was 50 blows per .2 ft. The soil encountered was a gray brown silty sand.

Subsurface conditions in the borrow areas. Hand auger holes A-6, A-7 and A-8 were performed in Borrow Area 1. The results of hand auger hole A-6 is fairly typical for the soils encountered in this borrow area. At this auger location, the surface stratum consists of approximately 2 1/2 ft. of soft yellow and brown silty clay. From a depth of 2.5 to 6 ft. below the existing surface is a firm yellow sandy clay which contains a trace of gravel. From 6 ft. to 15 ft. below the existing surface there is a red and brown micaceous sandy silt.

Auger holes A-1 and A-2 were conducted in Borrow Area 2. At auger location A-1, from a depth of 0 to 4 ft. below the existing surface, a red silty clay occurs, and from 4 ft. to 10 ft. below the existing surface is a red silt. At auger location A-2, from the surface to a depth of approximately 10 ft., a red and brown sandy silt exists.

In Borrow Area 3, auger holes A-3, A-4 and A-5 were performed. The results of the auger borings in borrow area three indicate a wide range in subsurface conditions. For example, at boring location A-3 there occurs approximately 1/2 ft. of gray topsoil which overlies a red sandy clay with a trace of gravel. From a depth of 5 to 8 ft. below the existing surface there occurs a sand and gravel with a trace of clay. From 8 to 15 ft. below the existing surface there occurs a red and brown clayey silt. At auger location A-5, the surface stratum, which is approximately 1 ft. thick, is a brown silty clay. This stratum overlies a red silty clay to a depth of approximately 10 ft.

The specific soil conditions encountered at each boring and auger location are indicated on the boring logs. The stratification lines represent the approximate boundary between soil types; in situ the transitions may be gradual.

## ANALYSES & RECOMMENDATIONS

For convenience the analyses and recommendations will be discussed under the following subheadings: Stability of slopes, Dam construction, and Construction problems.

Stability of slopes. Evaluation of the stability of a dam has two components -- slope stability analyses and analyses of the stability of the dam foundation. For the purpose of slope stability analyses, it was assumed that the cross section shown on a drawing prepared by Quible & Associates would be used in the construction of the dam. This drawing indicates that the width of the dam at the top will be approximately 20 ft. The downstream slope of the dam will be 1 vertical to 2.5 horizontal, and the upstream slope above the maximum pool elevation will be 1 vertical to 2.5 horizontal. Below the free board, this slope will be 1 vertical to 3 horizontal. On the upstream side of the dam, it is anticipated that a berm approximately 10 ft. wide will be constructed at the level of the aforementioned break in slope.

In order to determine the slope stability of this cross section, two triaxial tests were performed using on-site material suitable for construction of the dam. One triaxial test was performed using materials from samples marked A-3-1, A-3-2, A-4-1, and A-5. Based on laboratory compaction test data, these samples were thoroughly mixed, the moisture content adjusted to 16%, and the triaxial test specimen compacted to a dry density of 110 pcf. Before triaxial testing, the specimen was saturated and permeability tests conducted. Under a hydraulic gradient of 5 psi, the measured permeability was $2.3 \times 10^{-4}$ centimeters per second and under a hydraulic gradient of 10 psi the measured permeability was $3 \times 10^{-4}$ centimeters per second. The compacted soil sample for the second set of triaxial tests was composed by mixing equal parts of soils from auger holes A-2 and A-7. These samples were mixed at a moisture

... test indicated an effective cohesion intercept of 6 psi ... effective friction angle of 26 degrees. The results of the ... indicate an effective cohesion intercept ... and an effective friction angle of 30 degrees. The ... of these tests are presented in the appendix.

... analyses were performed for both rapid drawdown and ... conditions, i.e., with water at the maximum anticipated ... analysis, the Bishop method of slices was used. For the ... dam, the factors of safety for a deep circle, slope ... the water at maximum pool level, and steady state ... and the factor of safety for rapid draw- ... the upstream slope, the analysis for a ... drawdown situation indicates a factor ... failure for water at ... is approximately 2.3.

[illegible] of the dam, the analysis for a toe failure slide [illegible] of 4. In these analyses, the effects of [illegible] with the toe drain the factor of safety [illegible] is [illegible]ed. It would not be [illegible] against a failure of the upstream slope due [illegible] by flattening the slope. If such protection is desired, [illegible] approach would be to place a berm at the toe. If such [illegible], we would be pleased to design it for you.

Based on the logs from the seven borings made along the centerline of the dam, there should be no problem with regard to stability of the dam foundation. This assumes that the area covered by the dam will be properly stripped and all weak and compressible materials removed. Additional [illegible] will be given to the matter of stripping the site and removal of undesirable materials in later paragraphs. Some settlement of the dam will result, however, due to compression of the dam foundation under the weight of the dam. Settlement of the dam itself can also be expected due to its own weight. As a result of the variation in height of the dam along its centerline, settlement of the dam will be non-uniform and cracking will result if the magnitude of the differential settlement becomes too large. It is possible to reduce the detrimental effects due to cracking caused by differential settlement by using certain construction techniques. These techniques are covered in subsequent recommendations.

## DAM CONSTRUCTION

The Core. If a positive cutoff is required, the core of the dam must

extend at least 10 feet into the sound rock noted on the boring logs. The depth to this stratum varies, based on available subsurface information, from 30 ft. to more than 100 ft. below the existing ground surface, depending upon location. Sound rock was not encountered in boring B-7, which extended to a depth of 100 ft. The approximate depth to the surface of rock which would permit a positive cutoff is presented in Table 1, in the appendix. However, final elevations of the base of the cutoff trench, bottom of the core, must be confirmed in the field.

If seepage under the dam is permissible, i.e., a positive cutoff is not required, it is recommended that the area on which the core is to be placed be stripped to a minimum depth of 20 feet below the base of the dam, at all points. However, in the vicinity of boring B-4, where the overflow pipe will rest, it is recommended that the stripping for the central core cutoff continue until [illegible] the boring log.

The core should be constructed with a width of 40 percent of the maximum steady state water head. Sides of the excavation for the cutoff should be sloped to prevent cave ins. After the excavation for the core has been completed and before any fill material is placed, the excavated surfaces should be inspected by an [illegible] engineer.

Material used to [illegible] the core should be obtained from Borrow [illegible] clay as possible. Due to [illegible] in Borrow Area 3, [illegible] material to be placed [illegible] material [illegible] properties. If [illegible] of this type is not

available, it can be obtained from another site or some of the on-site material can be mixed with bentonite to provide a composite material sufficiently low in permeability. If the latter approach is chosen, additional laboratory testing will be required to establish the appropriate percentages of each material to be used. In addition, it would be desirable to have these materials mixed through a batch plant operation. However, with careful field control, satisfactory mixing could be achieved by the use of discs and harrows.

Earthwork. It is essential that fill used to construct the dam be placed in accordance with the minimum density criterion and in the range of moisture contents presented subsequently. Close control over all aspects of earth-moving and placement operations is essential. For this reason, it is recommended that full time inspection of all phases of earthwork [illegible] be inspected by an experienced soil engineer.

Considering variation in potential borrow material, before any material is placed in the dam, it should be approved by a soil engineer. Care should be taken to place the more cohesive borrow materials in the core of the dam, as cited earlier, and around the pipe which will pass through the dam. Such action will reduce the quantity of seepage through the dam and decrease the probability of "piping" occurring. In order to place the fill properly around the pipe, a great deal of care will be required. The soil for [illegible] the pipe will have to be placed by hand in order to insure proper placement. Some of the soils in the borrow areas are [illegible] should be used in the dam only if no other materials are available and then care should be exercised to see that they are placed in noncritical areas. It is recommended that the core be placed

in lifts not to exceed 9 in. in [illegible] the range of optimum moisture content plus 2 percent moisture. Fill to construct the core should be "[illegible]" [illegible] 3 [illegible] in accordance with [illegible] at least [illegible] percent of the [illegible] density [illegible] in [illegible] with [illegible] Specification D 1557-70.

The area on both sides of the core, within the base of the dam, should be stripped of all organic topsoil, tree stumps and any other unsuitable materials. For estimating purposes, [illegible] approximately 10 ft. of excavation will be required, subject to field verification. The [illegible] should be proof rolled with the [illegible] job site (minimum 25 ton loaded dump truck) and inspected. If any soft spots or unsuitable materials are encountered, they should be [illegible] and [illegible] [illegible] compacted to a minimum density of [illegible] of the maximum dry density in accordance with ASTM Specification D 1557-70.

Soil used to construct the dam may be obtained in the vicinity of auger holes A-2, A-3, A-4, A-5, and A-7. These materials should be placed [illegible] All soils should be compacted [illegible] the range of [illegible] moisture content plus 2 percent moisture. Soil used to construct the dam should be placed in lifts not [illegible] of the maximum dry density obtained in accordance with the procedures outlined in ASTM Specification D 1557-70.

Oversize cobbles should be removed from embankment materials prior to placement in the body of the dam. Assuming loose lift thickness of 9 in. will be used, no particle larger than 5 inches in maximum dimension should be placed in the dam.

All earthwork operations should be under the full time inspection of a soils engineer. Special attention should be addressed to construction in the area of the dam abutments, the cutoff trench, and the pipe.

Toe Drain. A toe drain having a minimum width height of 30 percent of the height of the dam should be provided. The purpose of this drain is 1) to reduce the pore water pressures in the downstream portion of the dam and hence to increase the stability of the downstream slope against sliding, and 2) to control any seepage water as it exits at the downstream portion of the dam in such a way that the water does not carry away particles of the embankment soil, i.e.,that piping does not develop. In order to function properly, the coefficient of permeability of the material of which the drain is constructed should have a permeability at least 10 to 100 times greater than that of the average embankment material. In addition, the pervious material from which the drain is constructed must meet the filter requirements for the embankment material. To provide for the adequate passage of water and to prevent clogging of the toe drain, material of which the toe drain is composed should conform to the following specifications:

1. The 15% size of the toe drain (filter) material (i.e., the particle size which is coarser than the finest 15% of the soil, $D_{15}$) should be at least five times as large as the $D_{15}$ size of the soil being protected.

2. The $D_{15}$ size of the toe drain (filter) material should not be larger than five times the $D_{85}$ size of the protected soil.

3. The gradation curve of the toe drain (filter) material should have roughly the same shape as the gradation curve of the protected soil.

4. Filters should not contain more than about 5% of fines passing the No. 200 sieve, and the fines should be cohesionless.

In the appendix are presented gradation curves for the proposed filter material and curves representing the range of materials to be used in construction of the embankment. The proposed filter material contains approximately 10 percent of particles finer than the No. 200 U.S. Standard Sieve (minimum dimension of 0.074 mm). However, 5 percent of this size is considered the upper bound for satisfactory performance of filter material. Therefore, the proposed filter material will have to be washed to meet this requirement. Any fines (material finer than the No. 200 sieve) remaining in the filter material should be cohesionless.

[illegible] should be [illegible] determinations. Satisfactory construction of the toe drain is especially critical due to the nature of the soil used in its construction. Considering the permeability data previously reported, seepage through the dam may be significant. Thus, the proper construction of the [illegible] to insure that the seepage line [illegible] to minimize the possibility of piping.

Camber. Available [illegible] constructed dams generally [illegible] height, depending on soil types. However, values [illegible] in excess of this amount have also been reported [illegible] compression of the dam foundation. Consequently, the [illegible] should be given sufficient camber to allow for post-construction compression of the [illegible] and

ment and foundation without a reduction in freeboard. Based on the fine-grained texture of the soils to be used in embankment construction and the consolidation test data presented in this report, a camber of at least three feet would appear to be necessary.

Freeboard. Sufficient freeboard must be provided so that there is no possibility whatever of the embankment being overtopped. The necessary freeboard is calculated by assuming that the maximum river flood will occur when the reservoir is full and the highest possible waves will develop at the same time. The minimum freeboard equals the computed head on the spillway crest at maximum flood discharge, plus 1.5 times the wave height (for runup on riprapped slopes), plus a safety factor. The safety factor generally varies between 2 and 10 ft. based on the dam height, the reliability of the data [illegible]

[illegible] erosion by wave action by the use of riprap. Riprap can be placed satisfactorily either at the time the [illegible] or after it is [illegible]

[illegible]

The pipe and Intake Structure. In order to minimize seepage along the pipe which is to run through the dam, the pipe should be equipped with collars which protrude a minimum of 6 inches from the outer surface of the pipe. At least one collar should be on each section of pipe. Great care must be exercised to insure that fill to surround the pipe is adequately placed.

There should be no foundation problem, with regards to the intake structure, if the recommendations presented in this report are followed. This assumes that the contact stress at the base of the foundation will not exceed 4000 psf and that prior to pouring concrete, the excavation for the foundation of the structure will be inspected by a soil engineer. Steel should be provided both top and bottom to allow for uplift on the base of the slab on grade.

## CONSTRUCTION PROBLEMS

During construction of the dam it will be necessary to divert the stream. This can be accomplished by use of a cofferdam.

Stream Diversion. The closure section requires special design considerations due to consideration of 1) provision of enough borrow materials and 2) the prevention of piping through differential settlement cracks. The closure section must be constructed rapidly to avoid overtopping -- work

cannot be stopped to search for additional construction materials or to process the water content of the soil. It must also be noted that this section, due to construction limitations, is especially susceptible to the development of differential settlement cracks. Consequently, consideration should be given to providing extra filter drains in the closure section in order to control any leaks developing through cracks.

The pool formed behind the cofferdam as the river is being diverted may flood the suitable available borrow areas. In such a case, adequate provision for this eventuality must be made a priori.

Groundwater. Measurement of the position of the groundwater table was made in only one boring, B-7. Such information is necessary for the assessment of potential problems relating to the need for dewatering during excavation and the planning of other aspects of construction. Based on the aforementioned water level reading, it appears that some positive technique for controlling groundwater will have to be provided during construction of the core and possibly the main body of the dam. In order to assess the extent of the problem, it would be necessary to install piezometers to measure water levels over a period of time. The earlier these can be installed the more valuable they will become.

The drain and filter material. The toe drain must be constructed of a suitable material with the characteristics previously described. Care must be taken to insure material used to construct the toe drain does not become contaminated with other materials.

The design has not been completed until the dam is built and the reservoir is in successful operation. During construction on almost every

job, problems arise which require design changes. For this reason, it is highly desirable that the designer also have charge of building the dam.

Post Construction Observations. In order to insure the dam is performing in the manner intended, it is recommended that observations of the dam be made periodically following completion of construction. This is especially necessary, at frequent intervals, for the first few years following construction to observe any leaks and/or cracking of the dam. Both of these, if remedial measures are not immediately taken, may lead to failure of the dam. Consequently, remedies should be instituted immediately upon observation of deficiencies.

The best assessment of dam performance can be made via instruments installed in the dam, e.g., piezometers, inclinometers, extensometers, settlement plates etc. If you desire, we would be pleased to design an instrumentation system for you.

## GENERAL

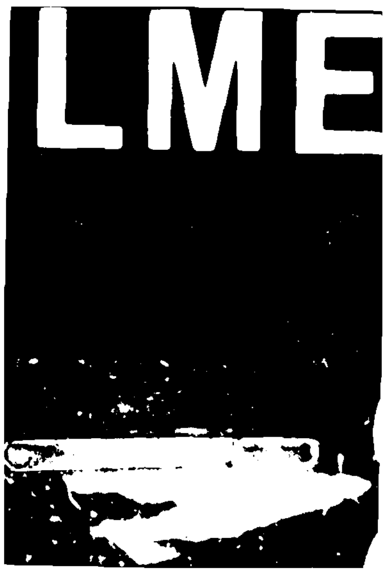
The analyses and recommendations submitted in this report are based upon the data obtained from 7 borings, 7 auger holes and 2 probe holes performed at locations indicated on the enclosed location diagram. This report does not reflect any variations which may occur between these borings and test holes. The nature and extent of variations between them may not become evident until the course of construction; if variations then become evident, it will be necessary for a re-evaluation of recommendations of this report to be made after performing on-site observations during construction and noting the characteristics of any variations. It is

recommended that the material at the base of the dam and core be inspected by an experienced soil engineer to ensure that these structures are placed upon suitable materials. It is also recommended that all stripping and controlled fill operations be inspected by an experienced soil engineer to ensure conformation with recommendations. If you wish, we would welcome the opportunity to perform any of the herein recommended inspection services for you during construction. In addition, we would be pleased to review the plans and specifications after they have been prepared for the project so that we might have the opportunity to comment upon the effect of soil conditions on the design.

This report has been prepared in order to aid in the evaluation of this site and to assist the architect and/or engineer in the design of the project based upon our understanding of the design details, criteria, and utilization of the structure as outlined herein. It is intended for use with regard to the specific project and locations described herein, and any changes in design or location should be brought to our attention so that we may determine how these changes may affect our recommendations.

## APPENDIX V - REFERENCES


1. Recommended Guidelines for Safety Inspection of Dams, Department of Army, Office of the Chief of Engineers, 46 pp.
2. Design of Small Dams, U. S. Department of Interior, Bureau of Reclamation, 1974, 816 pp.
3. Geologic Map of Virginia, Virginia Division of Mineral Resources, 1963.
4. HEC-1 Dam Break Version, Flood Hydrograph Package, Users Manual for Dam Safety Investigations, The Hydrologic Engineering Center, U. S. Army Corps of Engineers, September, 1978.
5. Hydrometeorological Report No. 33, U. S. Department of Commerce, Weather Bureau, U. S. Department of Army, Corps of Engineers, Washington, D. C., April, 1956.
6. Technical Paper No. 40, U. S. Department of Commerce, Weather Bureau, Washington, D. C., May, 1961.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